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SIMPLIFIED TECHNIQUES IN AIR POLLUTION MEASUREMENT[a]

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(Proc. Paper 1776)

INTRODUCTION

There is an increasing interest in pollution of the free atmosphere by airborne gases and aerosols and in evaluating the effects of such pollution on the environment. In many instances, control of air pollution is not undertaken until an environmental insult is proved. Evaluation of the insult generally means a survey which may take several forms. Testimony may be taken of claimed hazards; physiological and materials damage could be assessed; vegetation might be sampled and analyzed for suspected contaminants; or air samples may be collected. In the present state of scientific knowledge concerning air pollution, some information about the presence and concentration of suspected materials in the air is essential for correlation with other observations. There are numerous satisfactory methods available for accomplishing this end.

When faced with the need for sampling the atmosphere, one usually first considers some device for collecting a known quantity of air and having it analyzed in the instrument or in the laboratory. If put into practice, this supposedly simple procedure not only presents problems of a technical nature, but raises a question of suitability. Will the results justify the means? In many cases they most certainly will. In others, the fact that "X" micrograms of a given pollutant occur in every cubic meter of air sampled means little. Perhaps the effect of that concentration of pollutant is unknown at present. Or perhaps a concurrent meteorological study indicates that the sample was collected at a location which cannot be considered representative of the area.

A number of collection techniques have been devised for this purpose. For aerosols, they generally utilize the principles of filtration, electrostatic precipitation, thermal precipitation, or impingement. For gases and vapors,

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a. Presented at the April 1958 Industrial Health Conference, Atlantic City, N. J.

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absorption, adsorption, or freeze-out are frequently employed. The complete assembly usually includes a source of vacuum, a device for measuring the quantity of air sampled, and one or more of the above collectors. A timer or programmer may be used. Equipment of this type serves merely as a collector, and quantitative analysis must take place in the laboratory. The units have the advantage that by proper combination of collection technique and analytical procedure they can be made very specific for many polluting materials. Each complete unit may cost a few hundred dollars. Such units, however, are very expensive in time and manpower. For an extensive sampling program many man-hours may be required in making collections, maintaining the units, and performing the necessary analyses. Furthermore, the duration of sampling must be selected which will be as short as possible and still collect enough compound for analysis.(1) In some situations a duration of several hours will be required. A sampling period of given length will show the average concentration during that interval and peaks will not be noted. If it is known, for example, that exposure to a certain pollutant concentration for 15 minutes produces an undesirable condition, only that sampling duration or less could be used to define the cause and effect relationship.

To avoid many of these disadvantages, recording instruments have been developed for several of the common pollutants.(2) The ideal instrument, of course, would be a cheap, portable one which would continuously sample, analyze, and record instantaneous variations in the concentration of a pollutant and be specific only for that material over a wide range of concentrations.(3) Not only has the ideal not been attained, but devices approaching it are few in number. There are several reasons for this scarcity of instrumentation other than purely developmental.(2) There is at present little agreement as to the type of sampling which is required or even desirable for a given situation. For an extensive and continuous program, the available recording instruments may prove to be more economical since the analysis is performed in the unit. It should be borne in mind, however, that the existing instruments are expensive in first cost, many are temperamental in operation, and it may be difficult to eliminate interfering substances.

In setting up any sampling program, a number of factors must be considered.(1) Among these are:

1. Purpose of the samples.
2. Materials to be sampled.
3. Precision required.
4. Ranges of concentration of contaminant.
5. Limitations of analytical procedure.
6. Physical limitations and efficiency of collection device in conjunction with agent.
7. Location and number of sampling points.
8. Rate of sampling.
9. Duration of sampling period.
10. Meteorological conditions preceding and during sampling.

Consideration of these factors may lead to the conclusion that elaborate sampling devices are not necessary. The conclusion may also be reached that considerable preliminary information is required before a program can be planned.

For many purposes the convenience, precision, and characteristics of the previously described devices are not required. Studies of this type include determinations of the extent of pollution from a source, comparison of pollution in one area with that in another, evaluation of cumulative effects as in materials damage, identification of a source, and locality monitoring. In some instances, depending on the nature of the survey, it may seem desirable to obtain continuously several hundred simultaneous collections. The cost of a program of this type may be prohibitive in view of the cost of individual instruments. If vertical as well as ground-level samples are to be collected, standard samplers may be too heavy, bulky, or inconvenient to use. Practically all sampling programs are thus found to be a compromise between what is desirable and what is economical or convenient.

Simplified Techniques

An approach which is satisfactory for many purposes is to obtain cumulative indications over a period of time by simplified techniques. No quantitative measure of the concentration of polluting material per unit volume of air can be obtained by such methods but satisfactory information for many purposes may be secured. The unit cost of such techniques is practically insignificant when compared with more elaborate procedures and equipment. These techniques depend for their activity on having the natural movement of the air bring the pollutant in contact with the appropriate reagent. Results of most of these procedures are dependent upon wind velocity, atmospheric moisture, and air temperature. Although in some instances attempts have been made to refine and elaborate upon the basic techniques, much of the benefit may thus be lost. Generally, results obtained by simplified techniques correlate quite well with results obtained by other procedures. Simplified techniques may help to provide preliminary information on which to base decisions concerning an elaborate sampling program, or may be an end in themselves.

Sulfur Dioxide

Simplified techniques for aerosols, such as fall-out pans and jars, adhesive paper, and greased slides, are very familiar and will not be discussed here. Similar procedures for gases, however, are less familiar. The earliest of the dosimeter methods is applicable to one of the most common air pollutants. This is the lead peroxide candle method for estimating sulfur dioxide. The procedure was developed by Wilsdon and McConnell[4] for work done in England during the early 1930's. It is a cumulative method involving the rate of sulfate formation by exposing lead peroxide paste to sulfur dioxide. The candle which is used consists of a porcelain form about 10 centimeters in circumference. A 10 x 10 centimeter piece of cotton gauze is wrapped around the porcelain form as reinforcement and the active reagent applied. The active reagent is applied in the form of a paste consisting of 8 grams of lead peroxide in about 5 milliliters of gum mucilage. The mucilage is prepared by soaking 2 grams of gum tragacanth in 10 milliliters of absolute

alcohol, then dissolving with 190 milliliters of distilled water. The prepared candle, after drying in a desiccator, is ready for exposure. Exposure is in a standard shelter which protects the reactive surface from rain. After exposure for one month or longer, the candle is removed from the shelter and the area of the reactive surface measured. The coated fabric is stripped from the form by slitting with a razor and the amount of sulfate determined. A similar candle containing the gauze reinforcement, but which has not been exposed, is used as a control. The results of observations are reported as milligrams of sulfate per day per 100 square centimeters of lead peroxide.

Wilsdon and McConnel indicate that the rate of sulfate formation is proportional to the concentration of SO_2 in the atmosphere, at least up to 15 per cent conversion of the reactive material. From experiments in a wind tunnel, the rate of reaction was found to vary inversely as the 4th root of the wind velocity. An increase in temperature of 1°C will increase the reaction rate about 0.4 per cent. The reaction rate was also found to be increased considerably when the surface was wet. Conversion to lead sulfate was also found to be a function of the lead peroxide particle size. The authors noted a change in reaction rate with different batches of lead peroxide. In the work done in England, therefore, a large batch of lead peroxide sufficient to last for several years was obtained. This batch was ground to the proper size, mixed thoroughly, and a stock maintained at a central point.

The results obtained by this method were found to correlate quite well with information secured by other methods of analysis when the results were corrected for wind velocity and temperature. According to Parker and Richards,(5) units which were exposed for six months in London showed a mean rate of sulfation of 2.64 milligrams of sulfate per day per 100 square centimeters with a standard deviation from the mean of 7 per cent. It was estimated that the total sum of errors in the method will produce a standard deviation of about 10 per cent.

Gaseous Fluorides

Other pollutants for which simplified techniques are available are gaseous fluorides. One procedure reported by MacIntire(6) and his coworkers in Tennessee and by Wander(7) in Florida involves the use of the epiphyte, Tillandsia usneoides, which is commonly known as Spanish moss. Thie plant is not a parasite in that it obtains all of its nutritional requirements from the air. It does, however, need some support and is usually found on oak and other trees, although in some instances it may be found attached to power lines and fence posts. The plant is a cumulative sorbant for volatile fluorides. MacIntire exposed batches of the moss at various locations in an area suspected of being polluted by fluorides. Exposures were made at inside and outside locations for three months and showed substantial uptakes of fluoride of as much as six times the blank fluoride content. He concluded that the concentrations were indicative of the proximity to the source of suspected fluoride emission. The moss used in this study was brought into the area from nonfluoride areas.

Wander determined the fluoride content of native Spanish moss in the vicinity of a manufacturing plant producing triple superphosphate. The variation of fluoride content with distance from the plant was as follows;

Distance from plant in miles	F^- content in ppm
0.2	9400
1.5	688
2.5	688
3.5	888
4.0	132
5.0	110
6.0	60
7.0	100
20.0	100

He concluded that the source of fluoride emission could be pinpointed in this fashion. The deviation from the general trend of fluoride reduction with distance can be attributed to meteorological and topographical factors. For both cases, fluorides were determined by standard methods for vegetation.

In most areas of the United States, Spanish moss cannot be grown, except perhpas in the summertime. In areas where it can be grown, however, it serves as a useful indicator of the dispersion of gaseous fluorides.

A similar technique using limed filter papers in place of the epiphyte was reported by Miller(8) and coworkers from Washington. They used Whatman No. 1 filter paper soaked in a lime suspension consisting of 28 grams of low-fluoride lime per liter of water. The papers were dried and exposed in groups of six in shelters which permitted adequate air movement but protected the papers from the weather. Six treated papers were mounted in a rack in a vertical position, spaced 25 millimeters apart. In the work reported, a four-week exposure was used. In some instances, several sets of paper were exposed in various positions within the shelter and very little difference was found in the fluoride content. Following exposure, the papers were ashed and distilled. The results were reported in ppm fluoride, presumably on a dry weight basis.

The results were found to correlate very well with the fluoride content in adjacent gladiolus leaves and forage. The workers compared adsorption at four weeks with that at eight and it was found that the average of the eight-week exposure was 93 per cent of two successive four-week exposures. The method apparently serves as a record of the entire time of exposure and gives results which closely parallel the dose received by vegetation and other surfaces.

In both of the simplified techniques for fluoride collection, the results have been expressed in ppm on a dry weight basis. This may tend to confuse the casual reader if results of air sampling are also expressed as ppm. It might be better, at least in the case of limed paper, to express the results in a manner similar to that used in the lead peroxide candle method for sulfur dioxide.

Hydrogen Sulfide

Sewage treatment plant workers are responsible for another procedure which is useful in air pollution sampling.(9) This involves hydrogen sulfide estimation by the use of lead acetate impregnated papers or tiles. The latter is preferred. A lead acetate solution is prepared containing 500 milliliters of distilled water, 100 milliliters of glacial acetic acid, 450 grams of C.P.

lead acetate, and 350 milliliters of glycerine. Unglazed, white porcelain tiles of some convenient size are dipped in this solution, allowed to dry, and exposed in a place where they will be protected from rain. After exposure, the concentration of hydrogen sulfide can be estimated by comparing the exposed tiles with standards. Here again no quantitative estimate can be made but the relative exposure of various localities to hydrogen sulfide can be determined.

Ozone

Probably the most recent simplified technique in the field of air pollution is the use of rubber strips for the determination of ozone. A number of workers have recognized the effect of ozone on rubber and proposed it as an analytical tool. Bradley and Haagen-Smit[10] claim that rubber is a fairly specific reagent for ozone except that nitrogen dioxide and sunlight in air produce similar cracks. This may be due to the catalytic effect of nitrogen dioxide in producing ozone. They indicate that the action of ozone on rubber is influenced by (1) degree of stress; (2) nature of rubber compound; (3) concentration of ozone; (4) time of exposure; and (5) temperature. These workers used a standard rubber compound which was well-vulcanized and air dried for a few days. In their original work, the rubber was cut into strips 50 x 8 x 2 millimeters, folded into a loop, and bound at the open end. Thus, the elongation in the rubber varies from 0 to 100 per cent. The folded rubber unit was positioned in a tube and exposed to an air flow conducted past it at a known rate. The time of initial cracking - that is, when both edges and body exhibit cracking - was determined. The various batches of rubber compound were standardized by exposure to known concentrations of ozone at a flow rate of 1.5 liters per minute. Two parts per hundred million required 60 minutes for initial cracking, 26 pphm required 5 minutes, and 45 pphm required 3 minutes. After experience, an accuracy of $\pm$ 10 per cent can be expected in the observations. A similar procedure was described by Crabtree and Erickson.[11] They developed a standard curve for a given rubber formulation and provided for temperature corrections.

For use in routine field sampling, it is rather difficult to determine the exact time of initial cracking. The latest procedure, therefore, involves a measurement of the total crack depth after exposure to ozone. Rugg[12] first proposed the use of crack depth as an indication of ozone exposure. His proposal was based on a conclusion by Powell and Gough.[13] Rugg exposed stretched pieces of rubber to atmospheric ozone and measured the crack depth in a fresh section cut perpendicular to the cracks. The deepest cracks were assumed to be the active ones. The deepest crack and those within 10 per cent were measured under a microscope and averaged to get reported values. He investigated the effect of ozone concentration, temperature, exposure time, and elongation among other factors. He did not, however, apply the observations to air pollution measurements.

Haagen-Smit and Fox[14] applied the concept of total crack depth as a measure of ozone exposure. They used the sum of the depth of all visible cracks. They followed their previous procedure of using bent strips with the rubber being exhaustively extracted in carbon tetrachloride and dried. The prepared rubber strips were bent, tied, and exposed. After exposure, an examination was made on a newly cut surface 1 millimeter from the edge and perpendicular to the cracks. Under 100X magnification, the sum of the depths

of all cracks was taken as the significant value. For a 10-hour exposure, 1.0 millimeter total crack depth corresponds to 3 ppm of ozone.

A more refined device called an Ozonometer was developed by Beaty and June.(15) This simple device utilized the principle of creep in tension at constant stress due to rubber cracking. They found the rate of deflection was proportional to the ozone concentration with a reproducibility of ± 5 per cent.

CONCLUSION

Some of the techniques described have been used in the air pollution research projects at the University of Florida for such purposes as defining the geographical limits of a polluted area, confirming selected locations for volumetric samplers in a network, and estimating vertical concentration in conjunction with a volumetric sampler at ground level. They have been found quite satisfactory for these purposes and should be for many applications where dosimeter-type measurements are needed. Undoubtedly other reactants are described in the literature which could be applied as simplified techniques in air pollution measurements. More are needed for a variety of contaminants. Some of those described in this paper serve as collectors, with analysis taking place in the laboratory. Research should be directed toward more procedures which would make use of easily compared chemical or physical characteristics and would reduce the analytical costs. Additional research should be done in the area of the limitations and applications of simplified techniques. One word of caution, however, should be offered. In an effort to improve on procedures of this type, they should not be made so complicated as to lose their greatest attributes - simplicity and low cost.

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PROPOSED CHANGES IN EASTERN WATER USE POLICIES[a]

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(Proc. Paper 1777)

ABSTRACT

This paper considers the changing use of water in the east and compares with western states use. Also, consideration is given to the work being done on a "Model Water Use Act."

There seems to be very little doubt that the pre-eminent position of the United States, among the nations, has resulted in large measure from the quality of its people, a temperate climate, and an abundance of natural resources. Not in the least of these resources has been an ample supply of water of a useable quality. Ample up until now, that is, for now with our unprecedented growth in population and industry, even the humid Eastern part of the country is being pinched for enough water of a useable quality. Thus, we may be on the dawn of an era where water of useable quality will be this country's most critical natural resource.

This has long been the case in the seventeen Western States where water is not so plentiful. In these States, one only need compare values of lands which have water rights to the price of land which does not carry with it such rights. Because water shortages were encountered in this region almost from its time of original settlement, the Western States have developed a unique system of water law known as the "prior appropriation doctrine." Under this system priority of application of water to a beneficial use gives one the right to use the water so appropriated as long as the use is maintained. In a broad sense, this system may be characterized by the catchphrase - "First in

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a. Presented at the ASCE Convention in Chicago, Ill. February, 1958.

1. Chf., Interstate Enforcement Section Water Supply and Water Pollution Control Program Bureau of State Services, Public Health Service U.S. Department of Health, Education, and Welfare Washington D.C.

time - first in right." To illustrate, a junior appropriator located upstream cannot divert water for his own use to the detriment of the rights of a senior appropriator downstream. The doctrine of prior appropriation is written into State constitutions and State laws. Generally its administration results in specific allocation of waters to users under the jurisdiction of a State administrator.

In the Western States having the prior appropriation doctrine, the average annual rainfall is relatively small compared to the humid East. A major appropriative use is irrigated agriculture which is a highly consumptive use. It is estimated that two-thirds of the water applied to crops does not return to the watercourse. On the other hand, municipal and industrial users, which are the large users in the East and becoming increasingly so in the West, are generally non-consumptive users, ultimately returning more than 90 percent of the withdrawal to the surface waters of the State. In the 17 Western States, it is estimated that 92 percent of the water used is for irrigation and 3 percent is used by industry. In the 31 Eastern States, it is estimated that 81 percent of the water is used by industry and 3 percent for irrigation.

While the Western States have embraced the prior appropriation doctrine, their laws vary considerably from State to State. Only seven of the States (Arizona, Colorado, Idaho, Nevada, New Mexico, Utah, and Wyoming) do not have in their systems of water laws any elements of the riparian or reasonable use doctrine which is prevalent in the East. Since the prior appropriation doctrine of water rights deals with such a precious commodity in the Western States, it has been the subject of much litigation. In any particular jurisdiction, there are apt to be numerous judicial bench marks concerning water use which delineate the rights, duty, and privileges of the people of the State.

On the other hand, the Eastern and Midwestern States adopted the riparian doctrine which was embodied in the Common Law of England and practiced in the early colonies. This doctrine, in modified form, is presently applicable in all but one of the Eastern jurisdictions.

The riparian doctrine is based on the premise of running water. Under it, running water is not susceptible to unqualified ownership, and rights to such water are incidental to property ownership. Thus, only riparian owners, that is people whose land abuts a watercourse or have a watercourse running through their land, are entitled to riparian rights. Such persons, under the riparian doctrine, have a right to have a stream flow through their land in its natural state, undiminished in quantity or quality.

In the development of this doctrine and adjustment of conflicting riparian rights, it soon became evident that the riparian owner's right to the purity of a stream is not without limit. He possesses right to the flow and enjoyment of water but subject, however, to the similar rights of all the riparian owners to their reasonable enjoyment of the stream. It is only, therefore, in an unreasonable or unauthorized use of this common benefit that a legal action will lie. This modification of the riparian rights doctrine was enunciated in 1874 by Justice Cooley of Michigan as the "reasonable use doctrine" and has been utilized in that form by most of the Midwestern States. Under this doctrine, it is the right of the riparian owner to be free of unreasonable interference in the use of water and reasonable use is the only measure of riparian rights. The question of reasonableness is a question of fact to be decided on the merits of each individual case.

A characteristic of the eastern system of water rights is the absence of

frequent litigation. Water users have achieved an accommodation of their needs through private arrangements and surprisingly few of these have come before the courts. While this had the advantage of providing a flexible system of operation, it also has, over the period of time, created large areas of uncertainties without the guidelines provided by numerous judicial decisions. Recently, increasing and often conflicting demands of municipalities, industry, and agriculture for water have made eastern water interests acutely aware of the lack of certainty as to their rights in this field. In addition, recreation and conservation interests emphasize the complexity of the problem by their desires and requirements to keep the waters in the streams and maintain such waters at a sufficient quality to support fish and wildlife and permit other recreational uses.

Many of the eastern water users, particularly agriculturists, are reported to be reluctant to make investments in equipment to withdraw water from the stream without a clearer definition of their rights. In recognition of this problem, most of the States have established study commissions to reappraise their basic water laws.

Proposals for such reappraisals already have been presented to many State legislatures. For example, Mississippi has abandoned its traditional riparian doctrine and has substituted for it the prior appropriation doctrine, while Virginia has rejected a proposal to adopt the prior appropriation doctrine and has decided to go along with the riparian rights doctrine.

A composite of these proposals might best be examined by considering the work being done by the National Conference of Commissioners on Uniform State Laws which has undertaken the task of preparing a "Model Water Use Act" for consideration by the States. The Commissioners have assigned to the University of Michigan Law School the technical task of preparing the proposed legislation. Several drafts of a Model Water Use Act have been prepared by the Michigan Law School and it is now in the process of being revised again.

At a conference at Ann Arbor, Michigan, on September 4, 5, and 6, 1957, the principles underlying proposals in the Model Act were discussed by a large group of persons interested in the field of water. The proposed Act would:

1. Establish a single State administrative agency charged with the development, use, and control of all water resources of the State;
2. Vest administrative and legal responsibility for the control of water pollution in this new agency;
3. Establish a new system of water rights which would provide that:
   a. Where beneficial use is presently being made of surface water, existing rights would be frozen at the time of the enactment of the proposed Act;
   b. All other rights to use surface water would be governed by a permit system established and administered by the single agency; and
   c. Rights to use any ground water would be subject to a permit system established and administered by the single State agency.

These proposals are receiving considerable discussion among persons interested in the field of water resources.

A summary of the points around which the discussion centers follows:

1. The proposed Act assumes that eastern water rights law—the riparian or

reasonable use doctrine—should be changed. Many question the advisability of taking such a drastically irrevocable step before other possibilities have been thoroughly explored and tested. The existing system is one which is based on the common law and affects all the diverse relationships of persons with respect to the water use of the States. Any attempt to supplant this system with a single administrative code necessarily presents many sensitive and problem areas. In this connection, it may be pointed out that the prior appropriation doctrine was adopted in the West when the region was largely unsettled. In the long-settled eastern area where water rights have already been attached under a different system of law, many different questions are involved in applying any new unified doctrine.

2. The proposed Act may lack sufficiently precise legislative standards and criteria. For example, the provisions dealing with water quality permits authorize the proposed single agency to give such permits to the applicant making the "best" beneficial use of the water. No further substantive legislative direction is given. If a new system of water law is adopted in the East, consideration must be given as to degree of specificity with which the legislation will be drafted. If the legislation is not very specific, wide discretionary authority in vitally important areas will be vested in an administrative agency. This, of course, involves fundamental considerations of public policy. We do not wish to minimize, however, the difficulties in achieving specificity in the legislation. However, these difficulties may be symptomatic of the difficulty of finding equitable and understandable solutions of the problems involved. These same problems may prove to be equally difficult of solution by an administrative agency.

3. There may be considerable doubt as to the constitutional validity of taking water from a riparian owner without compensation and giving it to another permit holder. The validity of the taking is questionable even though the riparian owner is not making a present beneficial use of the water inasmuch as such owner has a present right to use such water. The constitutional issue posed may be stated as follows:

   Would such a taking represent a valid exercise of State police power or would it constitute a taking of private property without compensation?

   A simple example may serve to illustrate the problem.

   Suppose the case were one involving ownership of a tract of relatively undeveloped land which has a stream running through it. No beneficial use is being made of the water in the stream. If the water were diverted by an upstream owner, leaving a dry stream bed, would the owner of this property be deprived of a valuable property right and thus be entitled to compensation? And, would the owner be able to sell the property with a dry stream bed and get as much for it as if a lively stream were running through it ?

4. The proposed Act may not have given sufficient consideration to its effect on all water uses. These uses are: (1) municipal and domestic water supply; (2) industrial; (3) agricultural; (4) fish and wildlife propagation; (5) recreation; (6) navigation; (7) power development and transportation of wastes. The permit system proposed by the Model Act would seem to be designed to aid those users which withdraw water from the stream, such as industry or agriculture and possibly municipal and domestic water supply. The Model Act might not contain provisions sufficient to protect persons interested in the uses of water within the stream bed, i.e.,

dilution water for carrying away effluents from treatment plants, fish and wildlife, recreation and navigation.

5. The Model Act uses the old legal distinction of giving special consideration to domestic uses (personal needs and household purposes) of water. This may very well be too narrow a concept under modern conditions. For example, most public water supplies are servicing all the needs of municipalities and not merely domestic uses. However, agricultural representatives have indicated that many uses to which public water supplies are put, such as watering lawns and industrial uses, should not necessarily be given a higher priority than agricultural uses. The resolution of the problems in this area would appear to need careful and mature consideration.
6. Many persons believe that the characteristics of the individual drainage basins are so unique that plans for their development must be considered on an individual basis. Unless this is done, the premature issuance of permits may provide an unsatisfactory answer to the problem. For example, the Michigan Water Resources Commission, in a study dated August 1947 on the flow characteristics of representative Michigan Rivers, showed that flows of such rivers during the irrigation season are insufficient to meet the demands for irrigation water. Thus, if permits were issued, only a small portion of the potential demand could be met. The answer to this is obviously some form of water storage during period of maximum flow which occurs at a time of year other than the irrigation season. Because of the topography of the land, it is reported that even low-head dams are not feasible. A possible alternative may be the use of relatively large storage areas on individual farms. This, of course, would necessitate inundating much land now used for cultivation, and would involve basic evaluations of land, water, and farm policy. It would appear, however, that the premature issuance of permits to farms prior to the resolution of these many difficult problems may, in fact, deter the maximum use of the water resources of the State.
7. The proposed law is based on the assumption that it would be desirable to have a single operating agency to govern all water resources in a State. This may have been done without sufficient consideration being given to the establishment of a State water resource agency for coordinating, planning and development. Under the latter alternative, existing operating agencies would carry on functions in their specialized fields. For example, in thirty-three States, health departments or independent agencies administer State comprenensive water pollution control programs. Most of the comprehensive laws were enacted within the last decade. Many find it difficult to see what advantage would be gained by transferring the activities of these operating agencies to a single State agency concerned with all aspects of water resources. On the other hand, it may very well disrupt smooth and established operating programs within the States.

It seems to be generally agreed that more definite water resources policies are being evolved in the eastern States. The objectives of such a policy may be stated as: (1) the equitable apportionment of water among the many conflicting demands; (2) improving water quality; (3) stabilization of water flows and water supply. More and more eastern watercourses are becoming regulated streams. Along with this is the need for astute

comprehensive water quality management to insure that water is in a condition to meet the requirements of legitimate water users. As a prerequisite to the effective regulation of water quantity and the management of water quality, data must be assembled and analyzed concerning the potential useable water supply in the area. In addition, the uses of the existing water supply and a projection of requirements into the future must be made. Determinations in providing a supply of water of a useable quality in a particular area when needed will necessarily involve determinations and surveillance by State, local, and regional groups. The present choices which have been presented to the East are: (1) a revision of existing law into a single administrative code to be administered by a single State water resources agency; or, (2) the utilization of present water law (with such modifications as may prove to be needed) administered by the present existing operating agencies in cooperation with a State-wide coordination, planning and development agency. The resolution of these alternatives may well establish the pattern of water resource development in a large part of this country for the next 100 years.

Paper 1778 September, 1958 SA 5

Journal of the
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DESIGN OF CHICAGO'S CENTRAL FILTRATION PLANT

Fred G. Gordon[1], M. ASCE
(Proc. Paper 1778)

ABSTRACT

This paper consists of a general review of the design features of the central district filtration plant of Chicago, Illinois.

The Central District Filtration Plant, now under construction in a 60 acre cofferdam on Chicago's lake front north of Navy Pier, will have a nominal capacity of 960 million gallons per day based on a filter rating of two gallons per square foot per minute. The design however will permit operation at a filter rate of 3.5 gallons per square foot per minute equivalent to a plant capacity of 1,680 MGD.

General descriptions of the proposed design have appeared from time to time in the technical press. May I review these only to the extent required to serve as background for this paper. The general design is conventional including low lift pumps, mixing and settling basins, sand filters, clear wells and filtered water reservoir. One intake will be provided by a water tunnel connection to the existing lake section of the tunnel from the Dever crib. The second intake will be from port gates at the front of the plant. Tunnel connections for filtered water are under construction between the plant site and the existing tunnel systems serving the Central and North District pumping stations. All tunnels are in rock.

While, as stated, the plant is of conventional design its size presented problems not ordinarily encountered. These problems are the subject matter of this paper.

The general dimensions and location of the area to be occupied were fixed by ordinance. That in itself is quite a story and one for which the Supreme Court of the United States supplied the happy ending. The area is roughly

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rectangular with the length somewhat more than twice the width.

The need to concentrate operating personnel and facilities requiring the maximum attention within a minimum area near the center of the plant led to a design in which a central north and south work strip is flanked on the east and west by filters and settling basins. The filters are bisected by the long east and west axis of the plant and the settling basins fill out the four corners of the plant rectangle. This arrangement permitted the location of the control room, the control laboratory, the chemical feeder rooms, the wash water tanks and the administrative offices near the plant's major axes. North of the facilities in the central working strip are located the low lift and wash water pump room, the power company's substation, the boiler room, shops and store rooms. The south end of the strip includes chemical unloading facilities for railroad, truck or barge; chemical storage areas; and in the superstructure area general offices, laboratories and lecture rooms.

Low Lift Pumps

The selection of low lift pumps posed some interesting problems. The conditions which had to be met included pumpage rates from 400 to 1700 million gallons per day, a limitation by the power company on starting input to motors, varying heads on suction and discharge and the need to conserve space.

The maximum size of the pumping units was fixed by the largest motor which could be used - 1500 horsepower. In addition to the larger units smaller units were selected to provide a smooth transition from minimum to maximum pumpage. It was of course desirable to limit the number of the smaller size units to the minimum which would produce, with various pump combinations, a capacity pattern having suitable uniform increments.

Here the relationship between the number of small pumps and the relative capacities of the small and large units was the controlling factor. If only one smaller pump is used to produce uniform increments, it must have a capacity one-half that of the larger pumps. The increment, of course, is also one-half. If two smaller pumps are used to produce uniform increments, the capacity of each must be two-thirds the capacity of one larger pump. The increment then becomes one-third of the larger pump's capacity. If three smaller pumps are used to produce uniform increments, the capacity of each must be three-fourths of the capacity of one larger pump. Under these conditions the increment is one-fourth of the capacity of the larger pump.

Consideration of the foregoing relationships led to the selection of two small and six large units. The capacity of the large units at the design point is 260 MGD each and that of the two small units, 170 MGD each. With this equipment and proper pump selection, increases or decreases in pumpage rates can take place in 85 MGD steps. If filter rates are controlled by a fixed filtered water reservoir level, variations in consumption or pumpage are readily absorbed in a narrow band in the settling basin water levels.

At the South District Filtration Plant where pumpage increments are approximately 60 MGD and where the rated capacity is 320 MGD - or one third that of the Central District plant - similar plant operations to that just described have resulted in three to four pump changes a day with a settling basin water level held to approximately an 18-inch band.

The design head of the pumps, defined as the difference in elevation between the water levels in the suction and discharge channels was set at 25.5

feet. In determining this figure consideration was given to the number of hours of annual operation at various rates; the varying water level in the discharge channel; the friction loss in the raw water tunnel from the intake crib; and variations in lake level.

After an analysis of total annual costs for various types of pumping equipment, including structures, required auxiliary equipment and piping, it was found that the use of submerged vertical mixed flow pumps would result in the lowest annual costs. This type of pump - relatively new - is gaining rather wide acceptance as a large capacity, low head unit, particularly in the drainage and condenser fields.

In the design of the pumping station pumps were arranged on each side of a central operating aisle with provision for two future units. Suction and discharge channels were incorporated in double deck structures located below grade on each side of the pumping station building. A separate suction sump was provided for each pump.

It was recognized that a propeller type of pump is somewhat critical from the standpoint of submergence and suction channel location. Optimum conditions result when there is a uniform flow of water into the suction bell. Therefore in order to determine the most satisfactory clearance between the suction bell and the floor and back wall of the sump, model tests were run. The model used simulated two suction wells and the adjacent channel of the prototype. The two pumps used in the test were so mounted that they could be readily shifted horizontally or vertically between experiments. One of the pumps was of higher capacity than the size of the model dictated. The use of the larger pump was to exaggerate flow conditions in the model. In the prototype pumps down stream from the two pumps modeled will produce additional flow past the sumps of these pumps. In the model this flow was created by a circulating pump.

The criteria used to evaluate the pump sump performance were (a) pump efficiency, (b) perceptible vorticity about the pump suction bell and (c) pump noises. The results of the investigation indicated that optimum conditions were reached when the pump suction bell-to-floor clearance was 0.37 times the bell diameter and the bell to back wall clearance was 0.03 times the bell diameter used. It was also found by visual observation that vorticity at the sump gates was averted by the use of a projecting ledge at the gate ceiling.

Pump specifications required the manufacturer to furnish for each unit the pump, motor, butterfly valve on the pump discharge, and the short pipe connection between the valve and the wall of the discharge channel. It was specified that the pumps would be started unwatered by using compressed air to depress the water level below the pump impeller and that after the motor had attained full speed the butterfly valve would open.

Bids have been received for the pumping equipment, piping and valves and the award made to Fairbanks, Morse & Company on an evaluated bid basis. The guaranteed pump efficiency at the design point (head measured from pool to pool) was 90.0 percent for the 260 MGD pumps and 88.0 percent for the 170 MGD pumps. A field test to determine compliance with the overall efficiency guarantees is required. Because of the plant facilities available it is believed that such a test can be conducted with a high degree of accuracy. The successful bidder was required to build models of each size of pump and test them for efficiency, capacity, reverse runaway speed and freedom from cavitation before proceeding with the construction of the prototypes. Model tests were met successfully and the pumps are now under construction.

Characteristic curves furnished by the manufacturer indicate that the total capacity of the eight units when operating against a 20-ft. head will be 2375 MGD and when operating against a 30-ft. head, 1660 MGD.

Filter Units

Experience at the South District Filtration Plant was drawn on in sizing Central District filter units. South District filters each have a nominal capacity of 4 MGD. The influent and drain are both connected to a side gullet. The effluent and wash water connection is on the center line of the filter. The surface wash header (fixed system) is on the side of the unit opposite the gullet. Preliminary thinking on Central District design envisioned the use of the South District pattern with adjacent filter units converted into a double unit with a central gullet and a nominal capacity of 8 MGD. This would have required 120 filters. Final plans called for substantially the same design but with the filter length increased 25 percent, to bring the nominal capacity of a filter unit to 10 MGD, and the total number of filters to 96. One departure from the South District pattern was to place the drain at the back of the filter.

Filters are arranged with operating aisles at right angles to the main east and west gallery. Each side aisle serves six filters which constitute a unit for control and washing purposes.

Wash water pump capacity, wash water lines and wash water drains are designed to permit simultaneous washing of two filters but the filters must be in separate quadrants. While provision will be made for washing filters individually by hand, automatic control permitting the sequential washing of entire quadrants will normally be used. The type of control which will be specified is designed to eliminate all lost time in washing. The large number of filters and the possible frequency of wash under adverse conditions makes this requirement vital.

One characteristic of Lake Michigan water at Chicago is the prevalence at times of micro-organisms which cause short filter runs. The South District Filtration Plant, with 80 filters, has had in the past as many as 385 filter washes in 24 hours or nearly five washes per filter. Proportionately the number of washes for 96 filters would run to 462 in 24 hours. If two filters are washed simultaneously the average time of filter wash under the maximum conditions described would be approximately 6.2 minutes. The necessity for automatic washing thus becomes apparent.

Certain safety factors have been introduced in the design to reduce maintenance and insure proper filter washings. One of the most important is the use of a master wash water valve at the head of the gallery serving six-filter units. This valve is interlocked with each pair of back-wash valves for each filter so that it can not be opened unless the two backwash valves on the filter to be washed are open. Conversely it must be closed before the backwash valves will close. This prevents any wire drawing in the backwash valves and permits their operation with no unbalanced pressure. All wear is thus confined to the master wash water valve and provisions are made for ready servicing of its working parts. The master valve - in addition to the functions previously described - also serves as a shut-of or sectionalizing valve for each unit of six filters. A master surface wash valve will function in a similar manner to the master wash water valve.

Filter valves will be operated either hydraulically or electrically. Valves in filter pipe galleries will be operated electrically while those elsewhere will be actuated with hydraulic cylinders. Solenoid controls for hydraulic cylinders will be located in pipe galleries.

Bids for the wash water troughs, sand, gravel, and the under drain system were received with three alternative types of under drains specified. The low bid was on cast iron pipe laterals and the contract was awarded to the low bidder. This under drain system is the same as that in service at the South District plant.

Wash Water System

Two wash water tanks with a total capacity of 400,000 gallons are located on a floor 24 feet above the lips of the wash water troughs. Six wash water pumps, each with a capacity of approximately 20,000 gallons per minute, are located in the south end of the low lift pump room. The tanks float on the line and with proper control of cut-in points for the pumps will permit continuous pumping during automatic washing operations. Each quadrant will be served by a wash water controller.

Butterfly Valves

All filter valves and certain other valves throughout the plant will be of the rubber seated butterfly type. A decision to use such valves came only after long experimentation and considerable use at the South District Filtration Plant. Certain sectionalizing metal seated butterfly valves were installed in this plant at the time of its construction and have been in service about twelve years. In 1952 a 24-inch rubber seated butterfly valve (Henry Pratt Company) was installed for test purposes in the influent of one of the filters. It was down stream from the regular influent valve and was independent of washing operations in the filter. The valve, through the medium of a timer, was opened and closed three times every hour. In 1953, after 24,000 cycles, the valve was removed for inspection and was tested for water tightness. It was found bubble tight under air pressures up to 15 psi. In this test the valve is mounted in a test stand with the disc in a horizontal plane. Air is applied below the disc and leakage is observable by the passage of air bubbles through water standing on the disc. At 20 psi some bubbles began to appear near the point where the shaft passed through the rubber seat. At 40 psi bubbles at this location became violent and they also started to form about 90 degrees around the circumference from the first leakage point.

The disc of the test valve was cast iron with a hard chrome plated seating edge. The seat was pure gum rubber. Inspection of the disc showed it to be essentially clean on the seating edge. There were however signs indicating that the chrome plating was beginning to pit in the area that did not wipe the rubber surface on seating. There was some evidence of wear in the oilite shaft bearings at the inside edges closest to the rubber seat.

In addition to this accelerated test valve certain of the filters at South District were equipped with backwash and surface wash valves of the rubber seated butterfly type. These have proven entirely satisfactory. Since the tests just referred to were started the American Water Works Association has promulgated specifications for both metal seated and rubber seated

butterfly valves and such valves can now be purchased under these specifications with considerable competition.

Dehumidification

Dehumidification equipment will be used in pipe galleries and at certain other locations in the plant to check corrosion of pipe and equipment, reduce equipment failures, cut maintenance costs and improve working conditions. Here again the City's experience with similar installations has served as a guide in the Central District design. A pilot installation was made in the wash water pump room at the South District Filtration Plant in 1948. The results obtained were so satisfactory that equipment was installed to serve all of the plant's filter galleries. Following these installations similar equipment has been placed in the pipe galleries at two of the steam pumping stations. At the filtration plant the dry bulb temperature in the galleries varies from 40° to 72° F. during the year - depending primarily on the lake water temperature. The relative humidity varies between 45-55 percent, which range is sufficiently low to prevent condensation.

The equipment used is manufactured by the Surface Combustion Corporation. The operation is one in which the air to be dehumidified is passed through a spray of the absorbent - lithium chloride - which picks up moisture from the air. This in turn is regenerated by steam coils. The installation requires steam, cooling water, electric power and a means for circulating the dry air. In the case of filter galleries the layout is such that natural circuits can be used.

Chemical Handling

Chemicals will be received by railroad, truck or barge at the unloading area at the south end of the central strip. From here they will be transferred to storage tanks or bins and then moved as required to supply tanks or bins located above the chemical feeder rooms near the center of the plant.

With the exception of lime and activated carbon, all chemicals which are now considered for use at the Central District plant will be received in a liquid form or will be immediately converted to that form on arrivial at the plant. Activated carbon will be received in covered, hopper bottom cars and will be made into a slurry in concrete tanks located below the track. Lime will also be received in covered, hopper bottom cars and will be dumped into receiving bins of sufficient capacity to take a car load at one time. Conveyors will transport the lime from the receiving bins to the storage bins.

Track facilities for unloading chemicals include one receiving track for chlorine and two additional tracks for other chemicals. The three tracks are housed. The track for receiving chlorine shipments is adjacent to the evaporators, feeders and chlorine storage area, all of which are isolated from the rest of the plant by partition walls with provision for adequate ventilation. The chlorine track is equipped with track scales. The track adjacent to the chlorine track will be used for receiving activated carbon and lime. The third track has unloading facilities for other chemicals. Tracks are paved and most of the chemicals to be received in the third track area will normally come by truck. In addition to the three receiving tracks

which are housed there is a run-around track which facilitates handling of freight cars.

The use of chemicals in a liquid form has greatly simplified the storage and transportation problems. A notable example is the receipt of activated carbon in bulk, its conversion to a slurry, and its subsequent storage and application in that form. The possible variation in the use of activated carbon is well illustrated by operating experiences at the South District plant where as little as 1,600 pounds per day have been fed and as much as 80,000 pounds. In general storage facilities have been provided for 90 days usage for the various chemicals.

Filtered Water Reservoir

The filtered water reservoir is a reinforced concrete structure of conventional design. The feature which is of interest here is the provision made to meet the system hydraulic conditions imposed by the tunnel connections. A very short tunnel is required for the connection between the reservoir and the Central District tunnel system. However the tunnel connection to the North District tunnel system is nearly 5.3 miles long. It is desirable therefore to maintain a higher head on the latter tunnel than on the former if such a choice can be made. This is accomplished by a wall which divides the reservoir into two unequal parts and control gates at the inlets and outlets. The North Tunnel District whose water demands are about half those of the Central District is served by the smaller section of the reservoir. However by allowing filtered water from one half the filters to enter this smaller section and permitting the surplus to flow through openings at the top of the division wall this portion of the reservoir normally remains full of water. Any fluctuation in water levels then takes place in the area serving the Central District. The control works at the outlet shafts provides the necessary gates to permit the operation as indicated or the reservoir may be operated as a single unit. When required either or both sections may be dewatered without interrupting flow to the two tunnel districts.

ACKNOWLEDGMENTS

Obviously many people have been associated with the design and are and will be associated with the construction of this plant. These include in the Department of Water and Sewers: J. W. Jardine, Commissioner; W. W. DeBerard, Deputy Commissioner and Chief Water Engineer; H. H. Gerstein, Assistant Chief Water Engineer; and John R. Baylis, Engineer of Water Purification. Included in the Department of Public Works are: George DeMent, Commissioner; Dick Van Gorp, Chief Engineer; Walter E. Rasmus and J. Walter Grimm, Assistant Chief Engineers; George S. Salter, Chief Filtration Engineer; Ernst J. Stromback, Engineer of Water Works Construction; and Carl G. Riggenbach, Assistant Engineer of Water Works Construction.

The Consultants who reported on the project and who made recommendations as to the general design were Alvord, Burdick & Howson and Greeley and Hansen. Ralph B. Peck and Verne O. McClurg made independent reports on foundation conditions. Naess & Murphy are making architectural studies of the project and are preparing detailed architectural and structural plans for the central strip.

All work is proceeding under Richard J. Daley, Mayor.

Paper 1779 September, 1958 SA 5

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SED RESEARCH REPORT NO. 20

On — BOTTOM DEPOSITS IN A RIVER AND THEIR POTENTIAL EFFECTS ON DISSOLVED OXYGEN CONCENTRATIONS

By — The Sanitary Engineering Research Committee, Stream Pollution Section

From Research Data of — Warren H. Oldaker, Sanitary Biologist, Lawrence Experiment Station, Massachusetts Department of Public Health

Acknowledgement — The Sanitary Engineering Division gratefully recognizes the generosity and professional courtesy of Mr. Oldaker, and of the Massachusetts Department of Public Health, in making the data reported herein available for publication by the Society.

SYNOPSIS

The oxygen demand of organic materials accumulated on the bottom of a stream reduces the dissolved oxygen concentration in the flowing water. The oxygen demands of bottom deposits collected from the Connecticut River below Holyoke, Springfield, and West Springfield, are presented. Included is a discussion concerning the importance of evaluating the effects of such bottom deposits in any study of the pollution assimilative capacity of a stream.

Samples Collected From the Connecticut River

Deposits on the bottom of the seven-mile reach of the Connecticut River from just above the mouth of the Chicopee River to a point a short distance

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below the mouth of the Westfield River were sampled during September 1956. The locations of the individual samples collected are indicated in Fig. 1.

A Petersen dredge was used in sample collection. With such a sampler, normally only that material on, and within inches of, the surface of the deposit is sampled. In very soft deposits, deeper material might be picked up. No information is available on the depths of the deposits sampled.

Scouring at the mouth of the Chicopee River is known to occur quite frequently. At sampling sites 1, 2, 3, and 5, a major portion of the deposits is known to be trade wastes, rather than domestic sewage. At sites 6 to 10, the reverse is true, and the deposits here are known to be composed primarily of sewage solids.

Oxygen Demand of Samples

The sediment samples were each mixed in a Waring Blender immediately prior to the beginning of measurement of oxygen uptake in a respirometer. This process obviously mixes the older material, as measured by time since deposit, with the material more recently deposited, but the investigator felt that mixing would result in more representative data on the whole sample under study. Table 1 shows the relation of the rate of oxygen uptake to initial volatile solids.

Fig. 2 indicates the relative rates of demand, all of which are essentially linear with time, of the various sediments. Note that in general the higher rates of oxygen demand are shown by those sediments having the higher percentages of volatile solids. Mr. Oldaker feels the rates should be interpreted as being indicative of the relative magnitude of pollution occurring from point to point throughout the whole reach. He says the bottom sediments under study are subject to scour, especially at the confluence of the Chicopee and the Connecticut, and therefore the graph depicts the temporary and relative magnitudes of the effects of bottom deposits on oxygen concentrations.

Interpretation of Findings

The investigator points out that when Rudolfs[1] studied the Connecticut River in 1932, he found a range of 0.0409 to 0.1100 gr. oxygen per gram volatile solids (5-day Winkler at 20° C), or an average of 0.0754. Fair et al found a 5-day Winkler BOD of 0.331 gr. oxygen per gram volatile solids on a New England stream. According to these investigators,[2] fresh sewage sludge exerts a 5-day BOD of 0.3 to 0.5 gr. oxygen per gram volatile solids.

The report of the investigations, of which Mr. Oldaker's work is a part, "Connecticut River Study of 1956 with Reference to the Effects on Water Quality of the Proposed Enfield Power Project" states, "Indeed it is evident from a plot of the average results of the August 1956 survey in Figure XI that the low point in the dissolved oxygen concentration of approximately 3.75 parts per million occurred just below the Chicopee River which is where sludge banks are located. The September 1956 survey did not reveal any pronounced dissolved oxygen "sag" at all."

Perhaps the most important single point made by these studies is that in any study of the pollution assimilative capacity of a stream, the oxygen demand of bottom deposits must not be overlooked or underestimated. In

Table 1

Relation of Rate of Oxygen Uptake to Initial Volatile Solids

Sediment Sample Number	Moisture	Specific Gravity	Percent Total Solids as Volatile Solids	Cumulative Warburg Oxygen Uptake Grams Oxygen Per Gram Initial Volatile Solids				
	%			2 Days	4 Days	6 Days	8 Days	10 Days
1	77.7	1.14	3.53	.0328	.0467	.0578	.0689	.0817
2	59.1	1.29	2.50	.0360	.0564	.0752	.1015	.1235
3	88.7	1.04	14.4	.0577	.0842	.1158	.1532	.1797
4	97.4	1.04	11.7	.0606	.0939	.1318	.1689	.1982
5	92.7	1.07	6.1	.0500	.0620	.0680	.0768	.0908
6	58.7	1.22	6.73	.0144	.0241	.0315	.0378	.0442
7	58.4	1.23	6.08	.0184	.0281	.0352	.0426	.0490
8	45.3	1.33	2.97	.0249	.0347	.0428	.0504	.0573
9	78.0	1.00	9.68	.0449	.0809	.1168	.1398	.1555
10	66.3	1.22	4.33	.0198	.0300	.0394	.0462	.0532

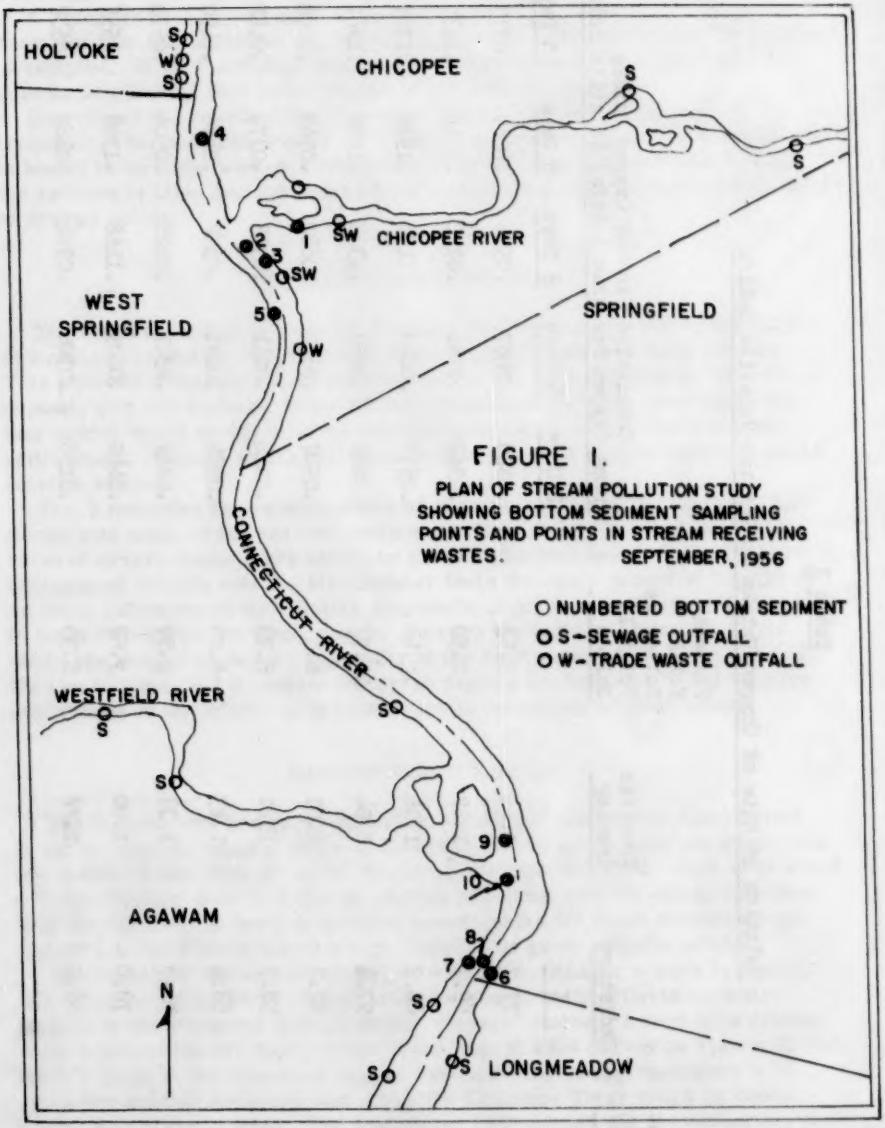

FIGURE 1.

PLAN OF STREAM POLLUTION STUDY SHOWING BOTTOM SEDIMENT SAMPLING POINTS AND POINTS IN STREAM RECEIVING WASTES. SEPTEMBER, 1956

○ NUMBERED BOTTOM SEDIMENT
○ S - SEWAGE OUTFALL
○ W - TRADE WASTE OUTFALL

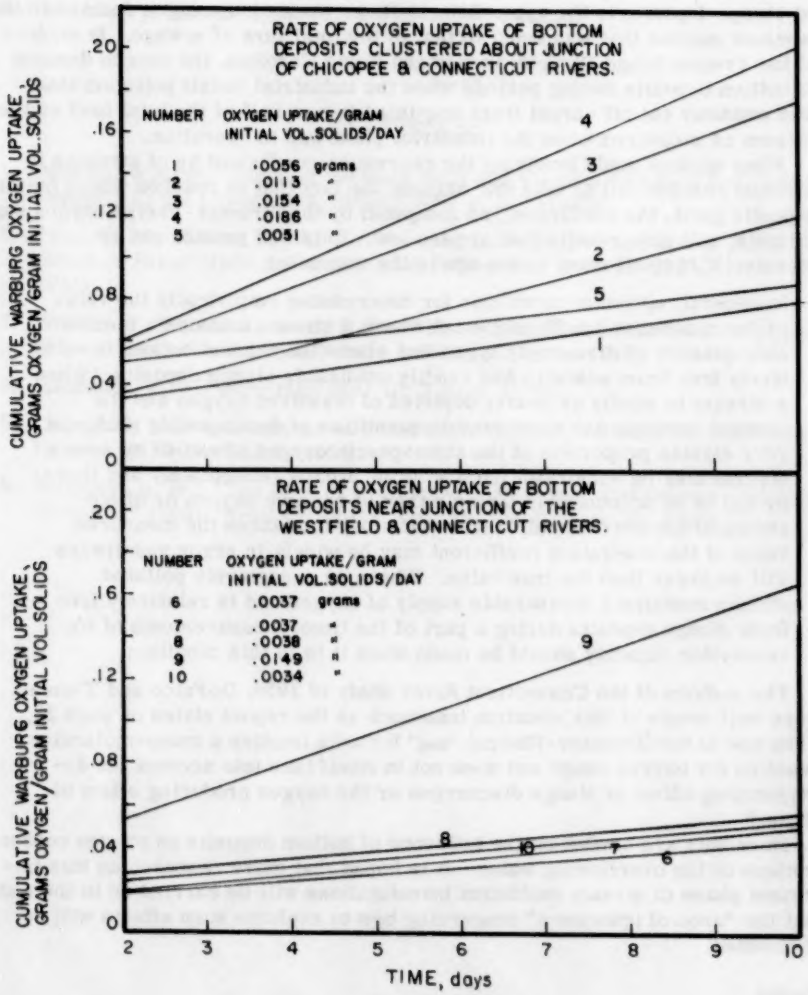
RATE OF OXYGEN UPTAKE OF BOTTOM DEPOSITS CLUSTERED ABOUT JUNCTION OF CHICOPEE & CONNECTICUT RIVERS.
NUMBER
OXYGEN UPTAKE/GRAM INITIAL VOL.SOLIDS/DAY
1 .0056 grams
2 .0113 "
3 .0154 "
4 .0186 "
5 .0051 "
CUMULATIVE WARBURG OXYGEN UPTAKE, GRAMS OXYGEN/GRAM INITIAL VOL.SOLIDS
RATE OF OXYGEN UPTAKE OF BOTTOM DEPOSITS NEAR JUNCTION OF THE WESTFIELD & CONNECTICUT RIVERS.
NUMBER
OXYGEN UPTAKE/GRAM INITIAL VOL.SOLIDS/DAY
6 .0037 grams
7 .0037 "
8 .0038 "
9 .0149 "
10 .0034 "
CUMULATIVE WARBURG OXYGEN UPTAKE, GRAMS OXYGEN/GRAM INITIAL VOL.SOLIDS
0
.04
.08
.12
.16
.20
2
3
4
5
6
7
8
9
10
TIME, days

FIGURE 2

studies of the Illinois River by Wisely and Klassen,[3] the oxygen demand of the bottom sludge in one reach was found to be 39 per cent, and in another reach 50 per cent, of the total demand in the reach. According to F. W. Mohlman,[4] the Board of Review of the Sanitary District of Chicago estimated sludge deposits in the upper Illinois River exerted an oxygen demand in the summer months that was equal to that of the total flow of sewage. In studies of the French Broad River[5] in western North Carolina, the oxygen demand of bottom deposits during periods when the industrial (total) pollution load was suddenly cut off varied from one-third to one-half of the total load on the stream as measured when the industrial plant was in operation.

Many studies made involving the reaeration coefficient k_2 of streams in polluted reaches fail to take into account the fact that in reaches where bottom deposits exist, the coefficient, as computed by the Streeter-Phelps oxygen sag formula, will give results that are too low. This was pointed out by Streeter[6] himself many years ago in the statement,

> In general, optimum conditions for determining empirically the value of the reaeration coefficient exist where a stream contains a measurable quantity of dissolved oxygen and where the channel bottom is relatively free from unstable and readily oxidizable sludge deposits. When a stream is wholly or nearly depleted of dissolved oxygen and its channel contains any considerable quantities of decomposing sludge, a very sizable proportion of the atmospheric oxygen absorbed by such a stream may be withdrawn from solution almost immediately and thereby fail to be accounted in terms either of reserve oxygen or of biochemical oxygen demand. Under such circumstances the measured value of the reaeration coefficient may be widely in error and always will be lower than the true value. Where an excessively polluted stream contains a measurable supply of oxygen and is relatively free from sludge deposits during a part of the time, measurements of its reaeration capacity should be made when it is in this condition.

The authors of the Connecticut River Study of 1956, DeFalco and Thoman, were well aware of this situation inasmuch as the report states on page 28, "The use of the (Streeter-Phelps) 'sag' formula implies a mono-molecular reaction for oxygen usage and does not in itself take into account the deoxygenating effect of sludge discharges or the oxygen producing effect of algae."

More data are needed on the influence of bottom deposits on oxygen concentrations of the overflowing water. It is hoped that more research on this important phase of stream sanitation investigations will be carried on to the end that the "area of ignorance" concerning how to evaluate such effects will be eliminated.

Credit

This research report is one of a series of professional contributions by the Committee on Sanitary Engineering Research:

E. R. Hendrickson) W. T. Ingram)	Air Pollution
M. A. Churchill	Stream Pollution
R. W. Bogan	Sewage
C. H. Hull	Water

	H. A. Faber	Public Health
	R. Stone	Solid Wastes
Chairman,	N. L. Nemerow	Industrial Wastes

and has been prepared by the Stream Pollution Section.

M. A. Churchill, Head

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RESISTANCE OF SEWAGE SLUDGE TO FLOW IN PIPES

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(Proc. Paper 1780)

SYNOPSIS

Due to heterogeneity and variability of material itself and complicated flow phenomena, the flow of sewage sludge in pipes defies direct application of existing hydraulic formulas for computing head loss. In this paper a new approach is proposed. First the flow is classified according to moisture content into two main types. With further subdivision of each type into laminar and turbulent stages, there are four categories of flow to be treated separately. For each, criteria are discussed and formulas are developed for practical application. Secondly by taking the moisture content and the origin of sludges as parameters a graphical method of statistical nature is suggested to determine the three physical properties of sewage sludge. With such empirical curves and rational formulas, it is believed a designer can predict the head loss of sewage sludge flowing in pipes under any conditions.

Nomencalture

C = Hazen-Williams' constant

D = Diameter of pipe, feet

G = Specific gravity

g = Acceleration due to gravity, feet per second per second

H = Head loss between two points in a pipe line, feet

L = Distance between two points in a pipe line, feet

M = Moisture content of sludge, per cent by weight

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1780 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 5, September, 1958.

1. Hydr. Engr., Hydr. Structures., Bogart and Childs, Cons. Engrs., New York, N. Y.

Q = Discharge of a pipe, cubic feet per second

R = Reynolds' number

r = Hydraulic radius, feet

S_P = Shearing stress at yielding point of a plastic material, called yield value, pounds per square foot.

$s = \frac{H}{L}$ = Hydraulic slope of a pipe line

V = Velocity, feet per second

V_{LC} = Lower critical velocity, feet per second

V_{UC} = Upper critical velocity, feet per second

W = Specific weight of water = 62.4 pounds per cubic foot

η = Coefficient of rigidity, pounds per foot per second

ρ = Specific weight of sludge, pounds per cubic foot

μ = Coefficient of viscosity, pounds per foot per second

INTRODUCTION

For piping and pumping sewage sludges, the conventional method of design based on empirical rules and curves, sometimes requiring "a factor of safety large enough to outweigh errors in assumptions," can hardly meet the various demands in actual practice. However, some valuable date and findings contributed from time to time by many authors from painstaking researches and elaborate experiments will be very helpful either in direct application or in further research if probable relations and coordinations can be established.

In order to facilitate the investigation, the following preliminary conditions will be observed: (1) there is no sound reason to amalgamate sewage sludge, clay slurry, wood pulp and drilling mud in one pack as a while, since nothing is definitely known to show their common bounds in either physical properties or flow behaviors. In this paper, it is intended to deal exclusively with the sewage sludge only. (2) There are many factors known to affect the physical properties of a sludge, but undoubtedly the outstanding one is the moisture content. It would be more efficient to single out this one alone in investigating the effects and correlations.

Characteristics of Sewage Sludge Flow and the Limiting Moisture Content M_L

From experiments of Bingham,[14] and Babbit and Caldwell,[1] it is known that sludge exhibits two types of flow of different physical characteristics depending on moisture content M. In a very dilute state, the solid particles are not in contact with one another, but are in free suspension. The presence of the suspended particles can only increase the density of the liquid and affect the viscosity to a negligible extent. The characteristic of flow is quite close to viscous liquid and far from plastic solid. Under this condition, the head loss is practically independent of moisture. This, for simplicity, is called

the flow in suspension. Conversely, in a less dilute state, when the solids concentration becomes sufficiently great to force the suspended particles into contact with each other, a noticeable resistance occurs to prevent them from sliding over one another, and there appears the so-called yield value of shearing stress which must be overcome before any movement can start. At the same time, the coefficient of rigidity η is increased over the corresponding coefficient of viscosity μ of the dispersion medium. Under such circumstance, the characteristic of flow is close to plastic and the head loss varies almost directly with the reduction of moisture M. This is called the plastic flow. Thus, it is clear that the head losses associated with the two types of flow are also entirely different in quality and quantity, and can be determined only by separate treatments. The dividing point between these two is called the limiting moisture content M_L, which is defined as the moisture content in percent where a measurable yield value S_Y first occurs. Hence, below M_L, the flow is plastic, and, above it, the flow is in suspension only. With this approach, it will be shown that the analysis of sludge flow can be simplified and some of the seemingly contradictory phenomena can be explained.

Critical Velocities and Reynolds Number R

It is generally recognized that in sludge flow, as in other fluid flow, there is a critical velocity which divides the flow into laminar and turbulent stages. As the two main types of flow mentioned above are different in character, the form of Reynolds is different.

With flow in suspension there is no yield value in the picture, and the Reynolds number takes the form of

$$R = \frac{\rho v D}{\eta} \longrightarrow \frac{\rho v D}{\mu} \qquad (1)$$

similar to that for water. ρ is the specific weight of the sludge which is proportional to moisture M. As to the value of η, some experiments show that it is very close to the corresponding value of water, i.e. μ.(2)

In plastic flow the apparent viscosity decreases with the increase in velocity(18) and, in a given range, it may be expressed as

$$\mu = \eta + \frac{16\, S_y D}{3v} \qquad (2)$$

and the corresponding Reynolds number becomes

$$R = \frac{\rho v D}{\mu} = \frac{3\rho D v^2}{3\eta v + 16 S_y D} \qquad (3)$$

There is no experimental determination of critical Reynolds number for sludges. By following Babbit and Caldwell and taking 2,000 as the lower and 3,000 as the upper limits of R, the critical velocities are:

For flow in suspension, where $M > M_L$;

$$v_{LC} = \frac{2000\mu}{\rho D} \tag{4}$$

$$v_{UC} = \frac{3000\mu}{\rho D} \tag{5}$$

For plactic flow, where $M < M_L$, from Babbit and Caldwell,

$$v_{LC} = \frac{1000\eta + 103\sqrt{94\eta^2 + D^2 S_y \rho}}{\rho D} \tag{6}$$

$$v_{UC} = \frac{1500\eta + 127\sqrt{140\eta^2 + D^2 S_y \rho}}{\rho D} \tag{7}$$

Statistical Determination of Sludge Properties

As shown above, the yield value S_y, the coefficient of rigidity η, and the specific weight ρ are the three fundamental quantities required in computing critical velocities and head losses. Unfortunately, little is known about them for most sludges. Also, it is repeatedly observed that these properties vary with moisture M, size, shape, gradation and character of the suspended particles, thixotripy, agitation, temperature of the material, etc. Moreover, these factors influence one another in complicated ways, some of which are not yet completely known. It seems no two sludges are ever identical. At the present it is almost impossible to formulate all of these factors into some simple functional relation for engineering practice. However, there is a way out of this situation; namely a statistical determination of the required quantities in empirical forms with respect to some parameters. In the problem at hand we may take the well-recognized factor of moisture content M, or its closely related solids concentration, in per cent by weight, as the principal index of sludge, and pack all others into the general term "origin or kind of sludge," such as "primary," "digested," and "digested from Imhoff tank," etc. By experience, we know the other important factor is the temperature, which is constantly varying and is seldom recorded; it will be neglected in this discussion.

Due to meagerness of the records, exact relationships cannot be established at the present. Such statistical plottings would be, after all, not mathematically precise. Nevertheless, the visual presentation of the experimental data, with trial curves of mean values, will help a designer make intelligent choice of proper constants to be used under various conditions. It should be mentioned here that most of the following values come from Babbit and Caldwell,(1) and Keefer.(15)

(A) G versus M. G is primarily used in computing specific weight as in $\rho = 62.4G$. For sewage sludges, the total range of variation is only 7%, and its determination can be made with confidence. For activated sludge, G = 1.007. The primary sludges are more erratic as we would expect, but the curve on Fig. 1 indicates a reasonable mean for them. The digested sludges have a cluster of points near G = 1.025, but the curve shows the general tendency. The three points from Imhoff tanks are on a perfectly smooth curve.

FIGURE 1

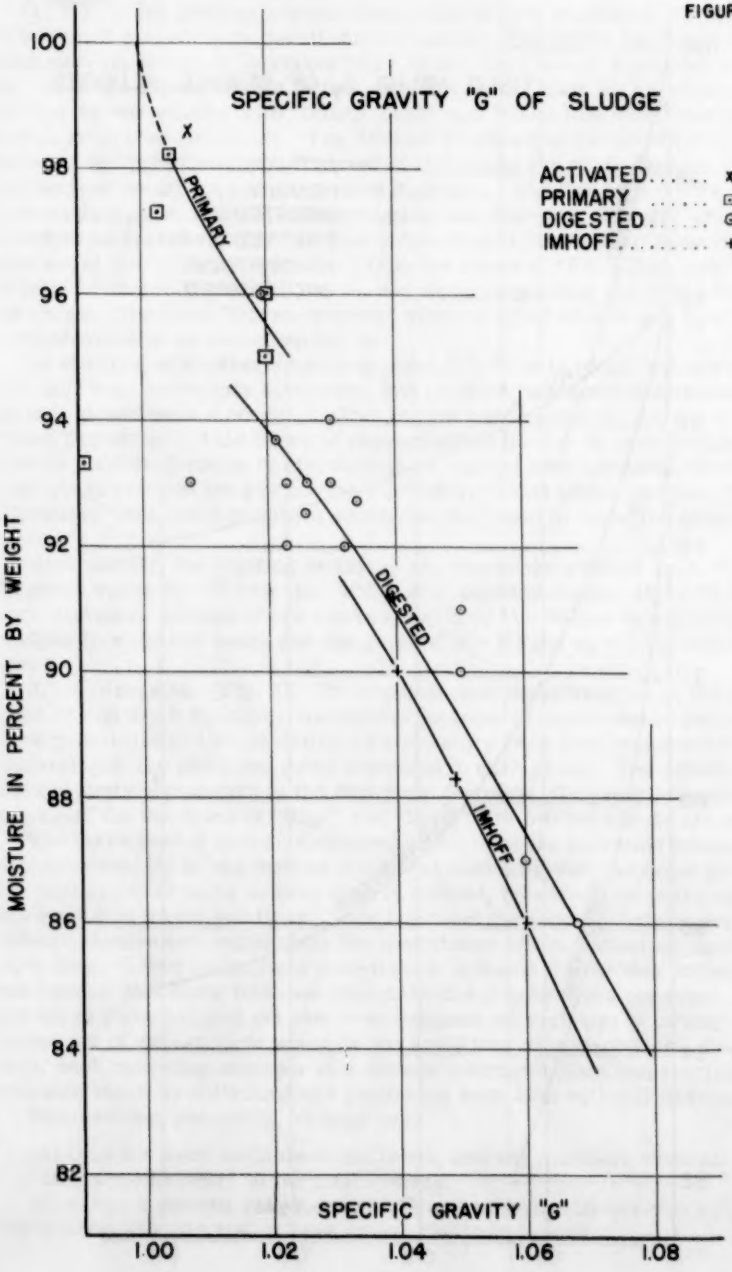
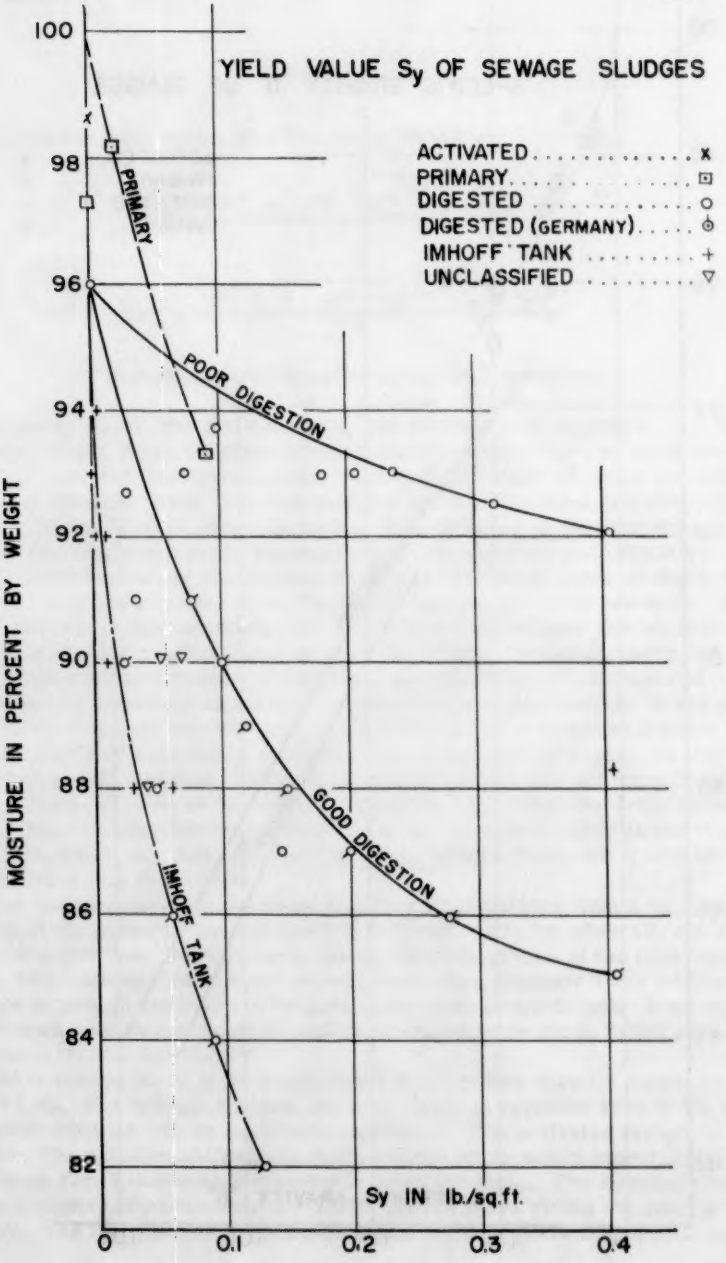

SPECIFIC GRAVITY "G" OF SLUDGE
ACTIVATED..... x
PRIMARY ..... ⊡
DIGESTED..... ⊙
IMHOFF..... +
PRIMARY
DIGESTED
IMHOFF
MOISTURE IN PERCENT BY WEIGHT
100
98
96
94
92
90
88
86
84
82
SPECIFIC GRAVITY "G"
1.00
1.02
1.04
1.06
1.08

FIGURE 2


YIELD VALUE S_y OF SEWAGE SLUDGES
ACTIVATED x
PRIMARY . . . . . ⊡
DIGESTED . . . . . ○
DIGESTED (GERMANY) . . . . . ⊙
IMHOFF TANK . . . . . +
UNCLASSIFIED . . . . . ▽
PRIMARY
POOR DIGESTION
GOOD DIGESTION
IMHOFF TANK
MOISTURE IN PERCENT BY WEIGHT
100
98
96
94
92
90
88
86
84
82
Sy IN lb./sq.ft.
0
0.1
0.2
0.3
0.4

(B) S_y versus M. S_y has an important part to play in calculating the head loss and the critical velocities.

On Fig. 2, the plots as a whole seem to be widely scattered, but the points from Imhoff tanks are on a well-defined curve. The points for digested sludges from Stuttgart, Germany build up two branches of a regular curve, the lower one falling almost in line with that from Imhoff tanks. Therefore, the two curves marked with "Imhoff Tank" and "Good Digestion" are regular and close to true conditions. The "Primary" based on two points may be taken as an approximation. The rest of the points run wild, perhaps due to variation of durations and methods of digestion. The five high points come from Indianapolis, Indiana, and Springfield and Danville, Illinois. One point of Imhoff tank and one of the ordinary digestion from Decatur, Illinois are also out of line with their groups. Thus the curve of "Poor Digestion" is an attempt to sketch in the high points, and should represent the upper limit of the range. The three curves converge at the point of $M = 96$ and $S_y = 0$ with a significance to be discussed below.

By checking with other empirical rules,[14,15] it is found that any digestion tank with up-to-date equipment, and of about one month's detention period, should have a condition close to that represented by the curve of "Good Digestion." This curve is recommended for use in such design. In case of shorter duration of digestion or of special arrangements, there is wide range between the two curves marked by "Good Digestion" and "Poor Digestion;" and sound judgment should be exercised to make the proper choice.

Indicentally, the limiting moisture M_L can be determined from Fig. 2 as the point where $S_y = 0$ cuts the curve. For digested sludge, $M_L = 96$ is a good approximation, because there are two points of $M = 96$ and $S_y = 0.0015$ for sludges from Imhoff tanks and one point of $M = 96$ and $S_y = 0$ for ordinary ones.

(C) η versus M. (Fig. 3). The experimental determination of the coefficient of rigidity η by either viscometer or from pressure-drop measurement is very delicate and its variation with moisture M is less pronounced than S_y. Accordingly, the plots are more scattered in each group. The reason for extraordinarily high values in the data from Stuttgart, Germany is unknown. However, the two lines of "High" and "Mean" are offered for design purposes.

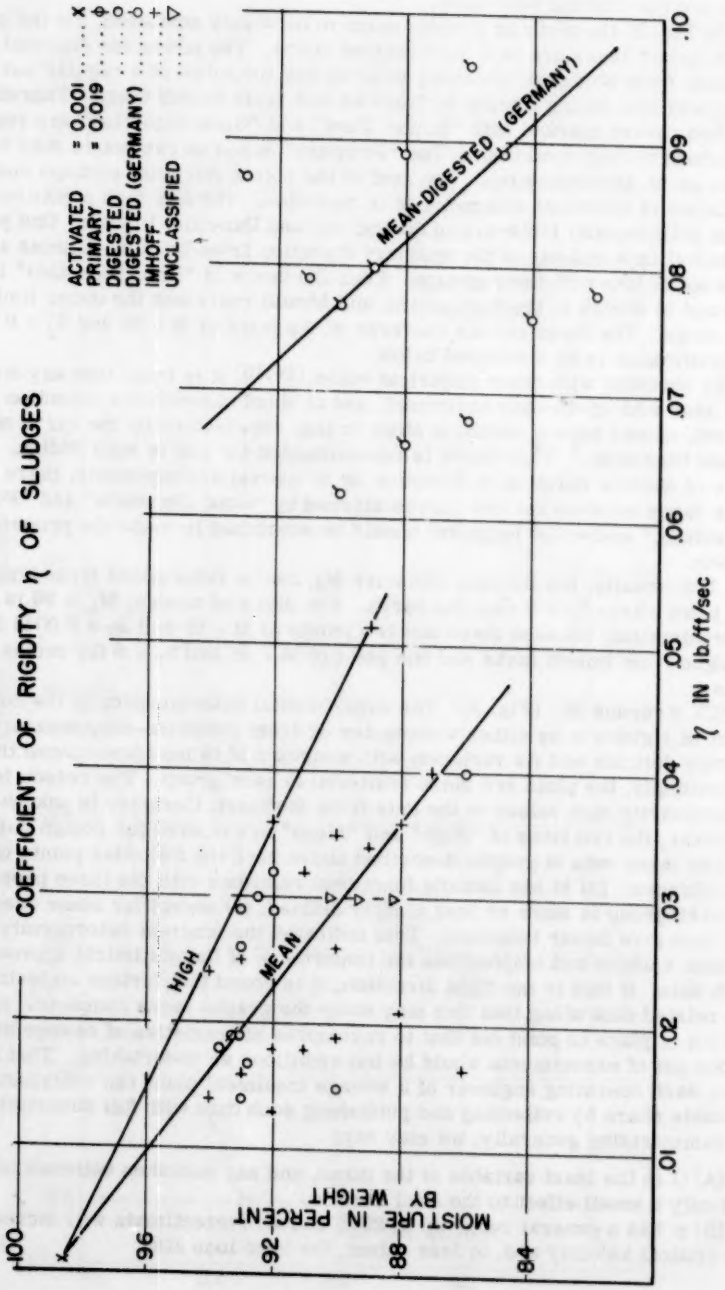
The three sets of graphs described above have the following points of significance: (a) M has definite functional relations with the three properties; (b) each group is more or less clearly defined; (c) except for some cases of S_y, none give linear relations. This indicates the inherent heterogenity of sewage sludges and emphasizes the importance of the statistical approach to such data. If this is the right direction, it is hoped that further collection of the related data along this line may make the graphs more complete. It is not out of place to point out that to encompass all varieties of sewage sludges in one set of experiments would be too ambitious an undertaking. That means, also, each operating engineer of a sewage treatment plant can contribute a valuable share by collecting and publishing such data with full description.

Summarizing generally, we may say:

(A) G is the least variable of the three, and any mistaken estimate of it has only a small effect to the final results.

(B) η has a general range up to 230% and an overestimate will increase the critical velocity and, to less extent, the head loss also.

FIGURE 3


COEFFICIENT OF RIGIDITY η OF SLUDGES
ACTIVATED = 0.001 x
PRIMARY = 0.019 ϕ
DIGESTED ○
DIGESTED (GERMANY) ○
IMHOFF +
UNCLASSIFIED ▽
MEAN-DIGESTED (GERMANY)
HIGH
MEAN
MOISTURE IN PERCENT BY WEIGHT
100
96
92
88
84
η IN lb/ft/sec
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01

(C) S_y has the widest range of variation, but fortunately the curves showing sludge origins are well defined.

It must be remembered that these statistics are based on records of the past. Any effort to bring the data up-to-date will improve and modify the empirical curves.

Head Losses of Flow in Suspension

Since the solid particles are free to move past one another and there is consequently no yield value to overcome, the reduction of moisture content only increases the specific weight ρ and, to a very small extent, the viscosity μ also. In fact, both are close to the values for water.

(A) In Laminar Stage. Here we have $S_y = 0$, and the equation for head loss becomes

$$\frac{H}{L} = \frac{\eta\, v}{\rho\, D^2} \tag{8}$$

in which both ρ and μ for the corresponding M can be determined from the curves.

(B) In Turbulent Stage. On this point, though little is known about the sewages, there is abundant reference material from similar problems in natural streams and laboratory tests. According to streck[9] and Winkel,[10] the head loss of flow in suspension may be put in the following form:

$$H_s = G^2 H_w \tag{9}$$

Where H_S is the head loss of flow in suspension with moisture M,

H_W is the corresponding head loss of pure water,

G is the specific gravity of the material.

If the above formula is applied to sewage sludge in the common range of such type of flow, the values may be shown as in Table 1.

TABLE I

G	1.01	1.02	1.03	1.04
$\frac{H_s - H_w}{H_w} = (G^2 - 1)$	0.02	0.04	0.06	0.08

The above figures are close to the findings of Hubbell.[13]

From experience, the head loss of such type of flow in both stages (Laminar and turbulent) is not far above the corresponding values of water.

Head Loss of Plastic Flow in Laminar Stage

This is by far the most common case in sludge flow, especially when moisture content is low. As explained above, the head loss is partly due to yield value and partly due to coefficient of rigidity, both of which are affected by the moisture M. As given by Babbit and Caldwell, it is in the form:

in terms of water head, $$\frac{H}{L} = \frac{16S_y}{3wD} + \frac{\eta V}{wD^2} \qquad (10)$$

in terms of sludge head, $$\frac{H}{L} = \frac{16S_y}{3\rho D} + \frac{\eta V}{\rho D^2} \qquad (11)$$

in which the values of S_y, η and ρ are determined from the curves mentioned before. Here are some points of practical importance to be noted. For any moisture below the limiting value there is a minimum head loss corresponding to V = 0 due to yield value alone, at which the flow is pending and below which no appreciable flow is possible. Therefore, any head acting below this minimum will just produce seepage,(1) and result in local increase in solids concentration. In other words, the moisture is just squeezed out of the sludge without noticeable movement of the material as a whole. Such conditions should be avoided in designing sludge pumps. Secondly, for given pipe size, the head loss increases with the first power of velocity in the laminar stage. Hence, as soon as the applied head is above the minimum a very small additional head is required to accelerate the flow to critical velocity. This checks with the experiments of Gregory(6) and others that the most economical velocity of sludge flow is the critical velocity, above which the head loss increases rapidly with the velocity. (See below). Moreover, as can be shown in Fig. 2 and by equation (10), the resistance to flow varies directly with concentration and inversely as the pipe diameter D. Consequently, it is wasteful to adopt small pipe sections or fittings, even for short lengths, in a pipe system of larger sizes, especially with sludges of low moisture. An empirical rule may be set for pipe economy that the minimum pipe diameter in inches should not be less than solids concentration in percent by weight (which is 100 - M).

Head Loss of Plastic Flow in Turbulent Stage

Operating data for this type of flow are contradictory and the published findings may be termed controversial. Due to variability of sludge itself and of the velocities used in experiments or encountered in field, the results are extremely divergent. Some investigators claim that the head loss is nearly the same as in pure water and independent of moisture; others maintain that it is about 1.5 to 8 times that of water, depending on moisture. Still others believe that sludge is basically heterogeneous and the usual hydraulic formulas cannot be applied to flow computations.

There is a theoretical background to all these hypotheses. If the flow is fully turbulent and the boundary layers are nominal only, the head loss is composed mainly of the impact of kinetic form which is proportional to $v^2/2g$ and the specific weight ρ. It will differ from that of water only by the effect

of ρ which, as shown on Fig. 1, varies very slightly with M; therefore it is independent of S_y and η. But this ideal condition of full turbulence occurs only when the moisture is above M_L as indicated in V-(B), or in some special sludges under high velocities as described by Babbit and Caldwell.(2) For the ordinary sludges the situation is not so simple. As the moisture is reduced below M_L, the critical velocities will be raised and the thickness of the boundary layers will be increased in proportion to moisture reduction. Furthermore, even at normal high velocities with turbulence in the outer circular ring, there may persist a central core with unit shearing stress less than S_y which will move along as a solid plug with a ring of another boundary layer. Thus the velocity distribution in a cross section and the outline of the boundary layers are quite different from the regular patterns of homogeneous liquids. Bingham, in referring to "hydraulic flow in the plastic state" says: "So far as known to the author, no one has yet used rates of flow high enough to bring about eddy currents. . . ." Thus, from such complicated phenomena, uniform results can hardly be expected. However, the following outstanding experiments are worthy of further discussion:

(A) Babbitt and Caldwell's Experiment on Turbulent Flow.(2) In order to ensure comparable results, the authors tactfully limit the tested material to ball clay of (a) sludge particles remaining in suspension without appreciable settlement, (b) true plastic state and (c) particles not disintegrating nor exhibiting thixotropic properties. The one digested sludge used was circulated for two hours before measurement was taken. By using pump and power of other types then those usually used for sludges they carried tests to velocities (up to 35 fps) far above those normally encountered in practice. In conclusion, they specify "the sludge must be made up of finely divided substance dispersed in water and the velocity must be above the critical." Under such conditions it is shown the sludge flow follows the fundamental laws of true liquids. The head loss can be calculated by Darcy's formula with the friction factor "f" corresponding to Reynolds number $R = \frac{\rho VD}{\mu}$, where ρ is the specific weight of sludge, and μ is the coefficient of viscosity of the dispersion medium. The results are independent of M and S_y and close to those of water.

From the special composition of the sludges and the special physical conditions of the tests, it seems the material has increased its fluidity to such an extent that it no longer has plastic characteristics; the coefficient of rigidity η has decreased to the corresponding value of viscosity μ of water, and the head loss is so much absorbed by the kinetic energy impact that the effect of yield value S_y is completely overshadowed.

It is apparent that no actual sewage sludge under normal condition will ever approach the state set by Babbitt and Caldwell, so their method of computation thus developed is limited to cases which are comparable with those described by them.(2)

(B) Greenwich Experiments by Brisbin. In his paper,(3) Mr. Brisbin does not show the basic properties S_y, ρ, and η of his sludges, and apparently most of his flows are in laminar stage except one of V = 5.11 fps. By limiting the results to the tests in turbulent flow,(20) the corresponding C in Hazen-Williams' formula

$$v = 1.318Cr^{0.63}s^{0.54} \qquad (12)$$

are computed from the observed head losses in different moistures in per cent of C at M = 100. At the same time, the corresponding multiplier of water head loss is obtained as $(\frac{C}{C'})^{1.85}$. These are tabulated in Table 2, together with values developed by American Well Works Company for comparison.

TABLE 2

M Moisture Content Per Cent	C' % Of C At M = 100%	$\frac{100}{C'}$	Multiplier of Water Head Loss $(\frac{100}{C'})^{1.85}$	Multiplier of Water Head Loss From Curves Of American Well Works
100	100	1.00	100	1.00
98	80.5	1.24	1.49	-
97	-	-	-	1.30
96	62.8	1.59	2.37	1.55
95	-	-	-	1.65
94	50.5	1.98	3.54	1.75
91.5	37.6	2.66	6.11	-
90	33.6	2.98	7.54	-

This indicates that Brisbin's figures are higher for lower moisture. But it must be remembered that Brisbin used raw sludges specially thickened up to 16% of solids concentration. Thus, for digested sludges, the corresponding C' may be reduced. More experiments on digested sludges are needed. In any event, the tabulated values can be used with a free margin of safety.

Fig. 4 is plotted by taking C = 100 when M = 100. If a different value of C at M = 100 is used, the values from the curve must be multiplied by the ratio $\frac{C}{100}$.

Procedure for Head Loss Computation

After the nature and composition of the sludge are known or assumed from a given condition, the first step is to determine the type of flow. Empirically this can be done by the curves on Fig. 2. On the other hand, if experimental data on head loss measurement are on hand, or can easily be obtained, a plotting of $S_p = 15.6\ D\ \frac{H}{L}$ as ordinate and $\frac{V}{4D}$ as abscissa, with moisture as parameter, will determine M_L as the point where $\frac{4}{3} S_y$ first occurs as an intercept of the curve on the ordinate (see Fig. 5 and 11 of (1)). Next, the constants are chosen from curves on Fig. 1, 2 and 3.

Example. Given primary sludge, M = 95. It is found to be in plastic flow.

From Fig. 1, 2 and 3,

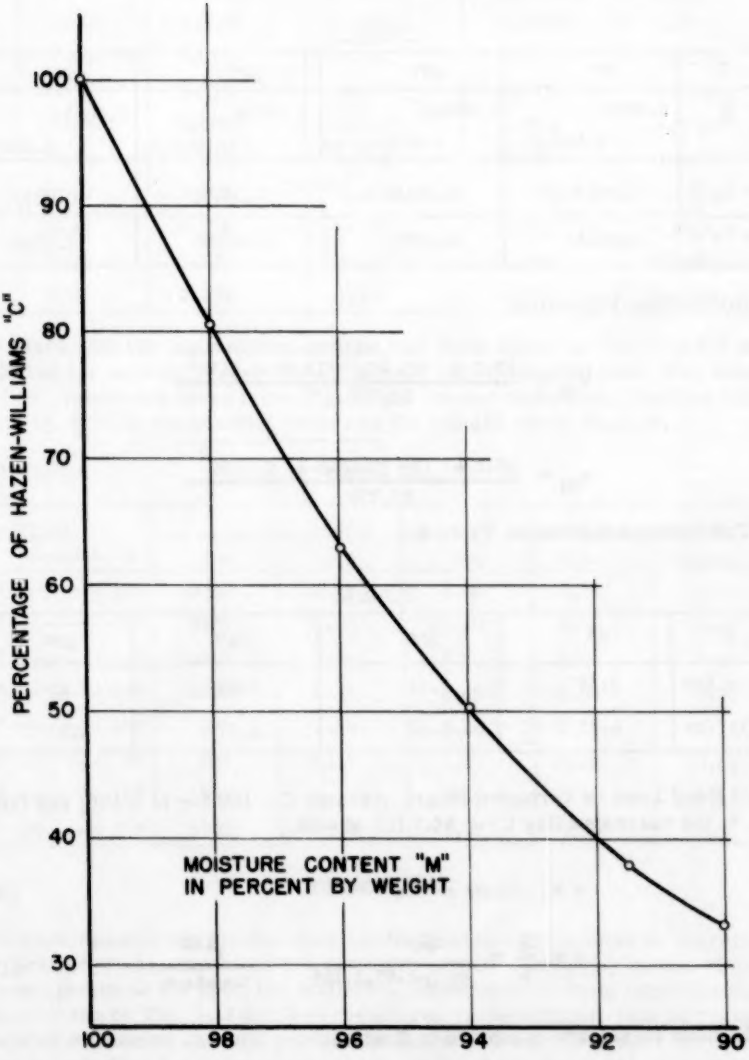
G = 1.022, ρ = 1.022 X 62.4 = 63.77 lb/sq. ft.

S_y = .065 lb/sq. ft.

η = .0127 lb/ft/sec.

FIGURE 4

VARIATION IN TURBULENT FLOW OF RAW SLUDGE



(a) Head Loss in Laminar Stage

$$\frac{H}{L} = \frac{0.00555}{D} + 0.000204\,\frac{V}{D^2} \tag{13}$$

in terms of water head.

The values are tabulated against the pipe diameter D in Table 3.

TABLE 3

D	8"	10"	14"	20"
$\frac{H}{L}$	0.00833 + 0.000458v	0.00666 + 0.000293v	0.00476 + 0.000149v	0.00333 + 0.000073v
v = 0, $\frac{H}{L}$	0.00833	0.00666	0.00476	0.00333
v = 7, $\frac{H}{L}$	0.00115	0.00871	0.00580	0.00384

(b) Critical Velocities:

$$v_{LC} = \frac{12.7 + 103\sqrt{0.01513 + 4.145D^2}}{63.77D} \tag{14}$$

$$v_{UC} = \frac{19.1 + 127\sqrt{0.0226 + 4.145D^2}}{63.77D} \tag{15}$$

The values are given in Table 4.

TABLE 4

D	8"	10"	14"	20"
v_{LC} in fps	3.58	3.55	3.45	3.40
v_{UC} in fps	4.52	4.42	4.31	4.23

(c) Head Loss in Turbulent Stage. Assume C = 100 for M = 100, and from Fig. 4, the corresponding C' = 54.7 for M = 95.

$$v = 72.09\, r^{0.63} s^{0.54} \tag{16}$$

$$s = \frac{H}{L} = \frac{v^{1.85}}{72.09^{1.85} r^{1.165}} = \frac{v^{1.85}}{\text{Constant}} \tag{17}$$

The head losses are computed in Table 5.

TABLE 5

v fps	v1.85	D : 8" v1.85 / 338.9	D : 10" v1.85 / 439.6	D : 14" v1.85 / 650.5	D : 20" v1.85 / 986.3
1	1	0.00295	0.00227	0.00154	0.00101
2	3.605	0.0106	0.00818	0.00555	0.00365
3	7.632	0.0225	0.0173	0.0117	0.00774
4	12.98	0.0384	0.0295	0.0200	0.0132
5	19.64	0.0580	0.0447	0.0302	0.0199
6	27.51	0.0813	0.0625	0.0423	0.0279
7	36.60	0.108	0.0832	0.0562	0.0371
8	46.85	0.138	0.106	0.072	0.0474
9	58.25	0.172	0.133	0.0896	0.0591
10	70.80	0.209	0.161	0.109	0.0718

To facilitate the computation, graphs like those shown on Fig. 5 and 6 may be plotted for each problem with the critical velocities indicated. For laminar flow, values are taken from Fig. 5, and for turbulent flow, they are taken from Fig. 6. The computation form and the results are in Table 6.

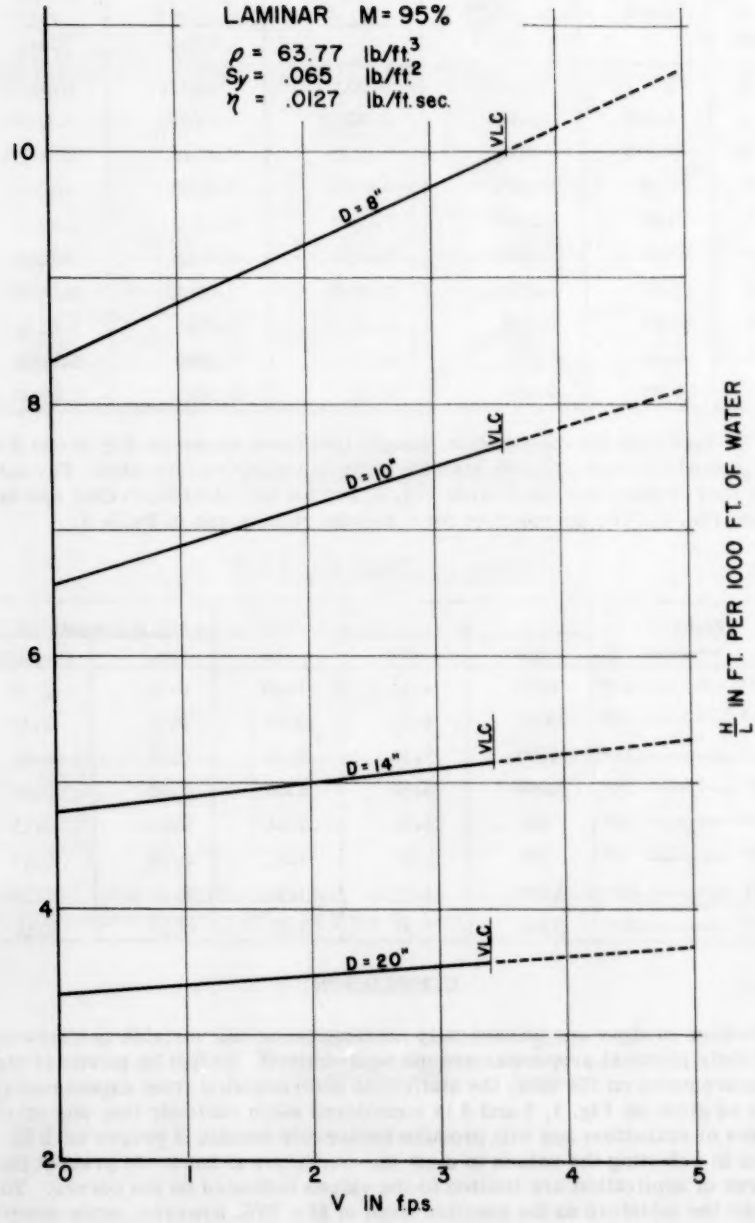
TABLE 6

Pipes L ——— D	Q gpm	Q cfs	fps	Head Losses, ft. Min.	Head Losses, ft. Operating
16' ——— 10"	2000	4.46	8.20	0.11	1.78
5' ——— 8"	2000	4.46	12.80	0.04	1.65
11' ——— 14"	4000	8.91	8.36	0.05	0.86
20' ——— 20"	4000	8.91	4.08	0.08	0.28
20' ——— 10"	600	1.34	2.44	0.13	0.15
50' ——— 8"	500	1.11	3.21	0.42	0.49
30' ——— 14"	1600	3.57	3.34	0.14	0.16
40' ——— 20"	3300	7.35	3.37	0.13	0.14

CONCLUSION

Sewage sludges are intrinsically heterogeneous and variable in character, and their physical properties are not well-defined. Except by means of direct measurements on the spot, the statistical determination from experimental data as given on Fig. 1, 2 and 3 is considered more realistic than any other means of evaluation and will produce reasonable results if proper care is taken in selecting the values to meet the conditions at hand. At present the ranges of application are limited to the values indicated on the curves. To extend the moisture to the possible limit of M = 75%, however, more observed data are required.

FIGURE 5


LAMINAR M = 95%
ρ = 63.77 lb/ft.³
Sy = .065 lb/ft.²
η = .0127 lb/ft. sec.
D = 8"
D = 10"
D = 14"
D = 20"
VLC
VLC
VLC
VLC
10
8
6
4
2
0
1
2
3
4
5
V IN fps
H/L IN FT. PER 1000 FT. OF WATER

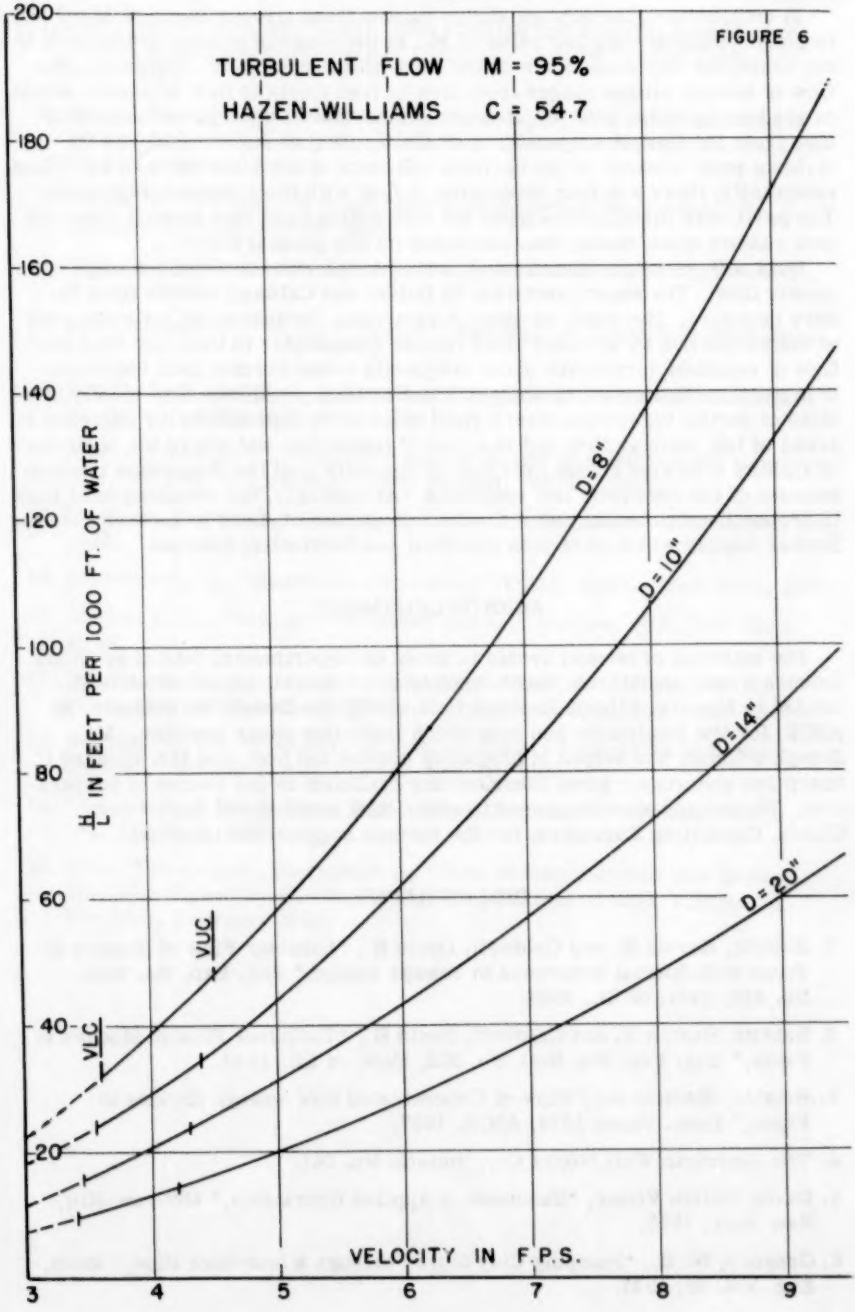


FIGURE 6
TURBULENT FLOW M = 95%
HAZEN-WILLIAMS C = 54.7
200
180
160
140
120
100
80
60
40
20
H/L IN FEET PER 1000 FT. OF WATER
D=8"
D=10"
D=14"
D=20"
VUC
VLC
VELOCITY IN F.P.S.
3
4
5
6
7
8
9

In composition, the sewage sludge ranges from viscous liquid at M = 100% to plastic solid at very low value of M. In the scope of normal practice, it is not exact, but approaches one or the other in the extremes. Similarily, the flow of sewage sludge ranges from flow of true liquid to flow of plastic solids in gradual variation with M. It is very important to distinguish the plastic flow from the flow of suspension with the limiting of moisture M_L as the dividing point because of the decisive influence of the yield value of S_y. Thus, essentially, there are four categories of flow with their distinct regimens. The particular formula developed for calculating head loss in each case will give results much closer than estimated by any general forms.

What still remains inconclusive is the phenomenon of turbulent stage of plastic flow. The experimentation by Babbit and Caldwell on this topic is very intensive. However, no other experiments (including the turbulent part of the recent one by Brisbin) yield results comparable to this, nor does such flow of complete turbulence occur frequently under normal field conditions. It is possible that ordinary sludges under normal conditions flow mostly in a state of partial turbulence where yield value of S_y still exerts its influence in areas of full laminar flow and in areas of transition; and where the coefficient of rigidity η is also higher than that of viscosity μ of the dispersion medium because of the relatively low velocities. Accordingly, the resulting head loss is proportional to solids concentration. In any event, here is another field for further inquiry which is of both practical and theoretical interest.

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Paper 1782 September, 1958 SA 5

Journal of the
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PLANNING THE FUTURE FOR CHICAGO'S WATER SYSTEM

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(Proc. Paper 1782)

ABSTRACT

The operation, design, and construction of the Chicago water works system were formerly carried on under the direction of the City Engineer in the Bureau of Engineering of the Department of Public Works. On January 1, 1953, all of the operating functions of the water system were placed under the Bureau of Water. Chicago is now eliminating possibilities of water shortages, and arrangements are being made to furnish water from the Lake Michigan source to suburban communities.

Many important changes have occurred in the organization of the Chicago water supply system since 1952. At that time the operation, design, and construction of the water works system were carried on under the direction of the City Engineer in the Bureau of Engineering of the Department of Public Works.

New Department

On January 1, 1953, a new Department of Water and Sewers was formed, in which all of the operating functions of the water system were placed under the Bureau of Water, headed by the Deputy Commissioner for Water and Chief Water Engineer. All water works design and construction functions, with the exception of water mains, were transferred to a newly created Bureau of Engineering in the Department of Public Works. In effect, the Department of Water and Sewers. The design and construction of water main extensions were retained in the Water Distribution Division of the new Bureau of Water.

Note: Discussion open until February 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1782 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 5, September, 1958.

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Reduction in Water Waste

It is believed a major achievement for the Chicago water system is that the annual daily average pumpage has increased only 22 million gallons per day (22 mgd) in the last 25 years. In 1956, the average pumpage was 1,032 mgd or 22 mgd more than 25 years previous (in 1932), when the average daily pumpage was 1,010 mgd. During this twenty-five year period, the population served had increased by 778,740 and considerable industrial expansion had occurred. Although the percentage of metered services had not changed materially, being 26.5 per cent in 1932 and 28.7 per cent in 1956, the metered sales had increased 217 mgd - (from 325 mgd in 1932 to 542 mgd in 1956). The unmetered water pumpage, which includes water supplied to assessed rate consumers, underground leakage and considerable use of water from fire hydrants—both authorized and unauthorized, dropped 195 mgd—(from 685 mgd in 1932 to 490 mgd in 1956). At the same time the average water pressure at curb elevation had been increased from 30 pounds per square inch (30 psi) to 38 psi.

This important accomplishment was brought about by an intensive program of finding and repairing underground leaks and house to house inspections for leaking fixtures, both carried out by the Water Distribution Division. Another important influence which has contributed to the reduction of unaccounted for water losses was the tearing down of large areas of substandard housing to make way for redevelopment projects and superhighways. These areas were largely unmetered and known to be exceedingly wasteful in water use. The new structures in these areas are supplied from metered connections.

Increase in Peak Demands

While the average yearly pumpage had remained practically the same during the twenty-five year period, in spite of increased population and industrial use, there was a definite increasing trend in the maximum day and maximum hour demands during peak hot weather usage. Chicago has no storage facilities in its distribution system. The intakes, tunnels, pumping stations, filtration plants, and distribution system must have sufficient capacity to supply good service during the peak instantaneous demands on the system. Prior to 1932, the peak day pumpage had been 1,287 mgd and the peak hour demand 1,466 mgd. By 1955 the all-time peak demands had increased 18 per cent to 1,513 mgd for the maximum day and 22 per cent to 1,788 mgd for the maximum hour.

It was evident that the water system needed expansion to meet the increasing peak demands.

Need for Expansion of System

Chicago had lagged somewhat in its water works construction during the period following World War II, principally because of difficulty in obtaining materials and because of inadequate funds available due to insufficient revenue brought in by the extremely low water rate existing in Chicago, which had not been changed since 1914 except for adjustments in discounts. This rate was the lowest of any large city in the United States. In July, 1951, the

water rates were increased by 50 per cent—from 8 cents to 12 cents per 1,000 gallons. Therefore, after 1951, funds became available for expansion of the water works construction.

The need for expansion of the water system was demonstrated in 1953, during the first year of operation of the new department, when the unusually long period of hot weather resulted in higher peak demands than had occurred in 1947. Inadequacies of the existing water supply facilities to meet the peak water demands in several areas of the city, as well as to suburban consumers, were revealed in a striking way. These inadequacies were principally in tunnel capacity, in pumping capacity, and in carrying capacity of the distribution feeder mains.

The 1953 experience brought about the realization of the need for major revision of our future planning, in order adequately to take care of the expanding use of water by the urban as well as suburban consumers, particularly by the rapidly growing metropolitan area, much of which was being served by well water supplies. In the metropolitan area, where heretofore the limited ground water supply had been adequate for all the needs of these communities, the increased requirements of the rapidly growing population were exhausting the available well water supply and lake water was the only unlimited source which could take care of these future needs. The realization of the shortness of available water supply in the metropolitan area caused much furore and alarm and demands were made to have Cook County, the Sanitary District of Chicago, and other governing agencies take over the responsibility for supplying water to the area.

Study of Water Supply Needs

Up to this time, no agency outside of the city organization had made a comprehensive survey of the water situation. To meet this need and to cover the whole field of engineering, finance and legal authority, the city, in 1954, retained the engineering firm of Alvord, Burdick and Howson. These consultants were to review the existing plans prepared by the city's engineering staff for providing adequate water service for Chicago and its suburban customers and also to determine the ability of the Chicago system to supply the water needs of the municipalities in the metropolitan area which might desire to be supplied from the Chicago system.

The consultants were asked to make detailed studies for the next twenty-five year period, up to the year 1980, of the adequacy of the city's capital improvement program for supplying satisfactory water service to the area supplied from the Chicago system; to survey the available water supplies in the metropolitan area and to estimate the growth in population and maximum water consumption rates for each of these communities; to outline additions to the water works required to provide adequate service; to make analysis of the Chicago water supply costs—present and future; to outline a practical plan for providing water service for the municipalities in the metropolitan area which might desire water service from Chicago; and to study the necessary water rates to bear the cost of required capital improvement for the Chicago system and to cover the increased cost of operation and maintenance.

The report, which was completed in September, 1955, predicted a rapid growth in the water supply demand for Chicago and suburbs, due not only to the expected increase in population but also to the modern trend for better

living conditions which results in more liberal use of lawn sprinkling systems, kitchen and laundry appliances, and air conditioning equipment, as well as the ordinary domestic and manufacturing water uses.

The report recommended modification of the Department's plan for water works capital improvements for the next five years, - 1956 - 1960, by rescheduling during this period projects which had previously been planned to be carried out after 1960. It also outlined a program for additional capital improvement construction during the following twenty-year period from 1961 to 1980 which would be needed if the ultimate anticipated water requirements of Chicago and its metropolitan area were to be satisfied. The required water works construction for the twenty-five years was estimated at close to $300 million.

The consultants found that Chicago could furnish more reliable water supply to the municipalities of the metropolitan area at a lower cost than could be provided by any other publicized plan proposed at that time. They determined that Chicago was well capable of furnishing Lake Michigan water to the entire metropolitan area, since the total supply required in this area was only a small portion of the total load on the Chicago system. The suburban customers required only 9 per cent of the total water pumped by the Chicago water system, and it was estimated that by 1980, if the entire metropolitan area as far west as the Fox River were to be supplied with Lake Michigan water, the total water supply taken would not exceed 25 per cent of Chicago's pumpage. However, the studies indicated that by 1980 it was likely that not more than 34 of the 85 municipalities not then supplies with Chicago water would require Lake Michigan water supply, and that therefore the estimated supply for the entire metropolitan area would probably not exceed 18 per cent of Chicago's total pumpage.

The consultants proposed a plan under which, if the proper legislative authority were provided, Chicago could finance and construct express feeder mains and booster stations outside the city limits to deliver water to the reservoirs of groups of adjoining municipalities in the metropolitan area. These municipalities could then distribute the water to their own systems. Under such a plan, Chicago could recover the cost of such construction, and operation, and maintenance of such facilities by a reasonable charge added to the base water rates covering the delivery of water to the city limits.

The consultants found that the revenue derived from the existing water rates would soon be insufficient to meet the cost of operation and maintenance of the water system, as well as the debt service on borrowings for the needed capital improvements. They therefore recommended that the water rates be increased.

Water Works Capital Improvements

The water works capital improvement construction program, which had already been expanded considerably with the availability of funds brought about by the water rate increase in July, 1951, was expanded to meet the recommendations of the consultants. Several projects which had been previously planned to be started after 1965 were advanced to be completed by 1960.

During the last five years, 1953-1957, the expenditures for water works improvements have totaled over $69 million, as shown in the summary

Table I

Summary of Water Works Capital Improvements

1953-1957

Year	Water Mains	Filtration Plants and Connecting Tunnels	Pumping Stations Tunnels and Misc.	TOTAL
1953	$4,167,511	$ 828,807	$ 2,433,703	$ 7,430,021
1954	5,575,837	2,751,413	3,026.461	11,353,711
1955	6,843,480	1,924,970	4,374.369	13,142.819
1956	6,161.933	5,760,189	4,931,226	16,853,348
1957 (est)	5,000,000	10,000,000	5,500,000	20,500,000
Total 1953-1957	$27,748,761	$21,265,379	$20,265,759	$69,279,899
Total 1948-1952	$14,166,176	$ 4,728,566	$ 6,036,174	$24,930.916

presented in Table I, whereas in the previous five years, 1948-1952, the total expenditure for improvements was slightly less than $25 million.

Feeder main construction was expanded greatly during the last five years —a total of 53.1 miles of mains, 24 inches in diameter and larger, were constructed in 1953-1957 as compared to a 6.7 miles in the previous five years.

During 1953-1957 a total of eleven pumps, having a combined rated capacity of 470 mgd were installed at the existing pumping stations, mostly to replace existing obsolete pumping units with efficient modern units—in some cases larger capacity than the original pumps. During this period the Fourteenth Street Pumping Station, having a rated capacity of 75 mgd was abandoned because of obsolencence. Seven modern high capacity boilers were installed to replace obsolete and inefficient units at steam operated stations; a completely new switch gear and transformer installation was completed at the Chicago Avenue Pumping Station; and a 30 million gallon reinforced concrete ground reservoir was constructed adjacent to the Western Avenue Pumping Station. These improvements have brought about more adequate water service to all parts of the city as well as to the suburban consumers.

The capital improvement program for the five years 1957-1961 contemplates an expenditure of $157,214,000 for water works construction. Under construction at the present time are the Central District Filtration Plant and connecting tunnels, which will have a rated normal capacity of 960 mgd and a peak capacity of 1,700 mgd. The total cost of this plant and tunnels is estimated at $95 million, and its construction, which was started in 1952, is scheduled to be completed in 1961 or early in 1962. The 79th Street Tunnel, on which construction was started in 1956 is 16 feet in diameter (Chicago Section), 4-1/2 miles long and is scheduled for completion in 1959. This project will add 600 mgd capacity to the existing south side tunnel system. Design has been started on a new 250 mgd Southwest Pumping Station and connecting tunnel, to be located at 84th Street and Keeler Avenue, with completion date set fo 1960. Design has also been started on two additional settling basin units and additional filters and low lift pumps to enlarge the South District

Filtration Plant. This will increase its rated capacity from 320 mgd to 480 mgd and the peak capacity to 800 mgd. Construction is also in progress on additional pump and boiler units at various pumping stations.

Considerable work has been carried on in the past few years in relocation of feeder mains and small mains because of the construction of superhighways in various parts of the city. Major main reconstruction will be required in the vicinity of these stations.

The modern trend of increased use of water by consumers for kitchen appliances, lawn sprinkling, and air conditioning, and the expansion of population and area which will be served by the Chicago system, will require continued expansion of the water system, in order to provide adequate service at all times. The consultants estimated that the 1980 water requirements for Chicago, as well as for communities in the metropolitan area, are: 1,490 mgd for average day; 2,410 mgd for maximum day; and 2,760 for the maximum hourly rate. It is the department's policy to program the improvements so as to have needed facilities completed and ready for service ahead of the development of the demands.

On May 1, 1957, a 33-1/3 per cent rate increase from 12 cents to 16 cents per 1,000 gallons was placed in effect in order to finance the debt service for capital improvements construction and provide for increased cost of operation and maintenance of the water system.

The state legislature, in 1957, passed two bills which now authorize Chicago to build and operate water works facilities as far as 35 miles beyond the city limits, to permit Chicago to wholesale water to communities in the metropolitan area. Chicago would be permitted to charge an extra rate above its standard metered rate to cover fixed charges and operating costs for constructing and operating these outside facilities. Previously Chicago was not permitted to construct facilities outside of the city limits. Chicago is still required to furnish water at the city limits to any community in the Sanitary District of Chicago, at a rate not higher than the same rate charged to the metered consumers inside Chicago.

Summing up, Chicago is well on the way to eliminating any water shortage scare even on the hottest days. Suburban customers can feel well assured that the facilities are available to bring water from the inexhaustible Lake Michigan source to their own reservoirs or pumping units. The Chicago consumer will continue to receive an adequate, sparkling clear potable water at proper pressures.

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CAPACITY CRITERIA FOR REFUSE INCINERATION

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(Proc. Paper 1783)

SYNOPSIS

Changes in accepted methods of stating refuse incinerating capacities are proposed, chiefly to distinguish plant capacity from furnace capacity. The importance of the class of refuse to be burned is stressed, and certain design criteria, based roughly on Btu loadings, are suggested.

INTRODUCTION

It is the writer's observation that presently accepted methods of determining and of stating the capacity of refuse incinerating plants lead to considerable misunderstanding and improper comparison of data concerning such plants. This refers to such data as construction costs, operating costs, sizes of plant components, operating rates and results and the like.

The difficulties arise from three principal sources:

a) the universal practice of stating plant capacities in terms of tons per 24 hours, without regard to the ability of the plant to operate 24 hours per day,

b) failure to state or to consider the characteristics of the refuse which the plant is designed to burn at rated capacity.

c) The tendency to overrate incinerator furnaces, using the result of short period tests under optimum conditions.

Stating Plant Capacities

The use of a 24-hour period for stating incinerating capacities has no logical basis. The pertinent period (for plant capacities) is the week, since

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a weekly shutdown for cleaning and maintenance is necessary for proper operation, and it is only at the week's end that the storage bin need be emptied. The plant capacity depends, therefore, not only on the rate at which the furnaces can operate, but on the number of hours per week during which they can operate, as limited by the refuse storage capacity available.

Similarly it is suggested that a more logical procedure for rating furnaces is in terms of tons (or pounds) per hour. A floor-dump plant with no storage available and operating five 8-hour days would have a weekly capacity only one third of that of a crane-and-bin plant with the same furnace capacity but with storage to permit continuous operation for five days. Intermediate storage provisions, as for two-shift operations, can be used.

Storage capacity has sometimes been less than adequate for the purpose intended. The procedure used in a recent design for determining storage bin volume will illustrate some of the factors involved.

There were excellent records of the daily and seasonal variations in refuse (garbage and combustible rubbish) quantities, which indicated a maximum week of 135 per cent of the annual average, with collections on 5 days, and of 125 per cent of the average when collections are limited to 4 days due to a holiday. Daily variations within the maximum weeks are as follows:

Day	Refuse quantity - per cent of total for week	
	5-day week	4-day week
1	22.2	26.6
2	20.5	25.3
3	20.5	25.0
4	19.1	23.1
5	17.7	--

It was concluded to base the furnace capacity on 5-day operation during the maximum 5-day collection week and to provide sufficient storage capacity to permit overtime operation during the occasional 4-day collection week which coincides with maximum refuse quantities. A straight-forward computation indicates the required storage as about 29 per cent of the total quantity for the maximum 4-day week, or about 32 hours of furnace capacity. Had provision been made only for the 5-day collection week the required storage would have been about 22 hours of furnace capacity.

Measurements of volume and weight in the storage bin (the project involved expansion of an existing plant) led to the adoption of a unit weight in the bin of 300 pounds per cubic yard for periods of maximum refuse quantities. Of the resulting computed storage volume, 75 per cent was provided below the level of the tipping floor, the remaining volume being available above that level by rehandling the refuse with the cranes.

The question then arises as to what is the proper figure to be assigned as the capacity of this plant, whose furnace capacity is 23 tons per hour. It would be possible to maintain that the excess storage volume makes possible overtime operation of the plant during a 5-day collection week to the extent of 19 hours, resulting in a plant capacity of 139 times 23, or 3,200 tons per week. It is suggested, however, that the proper rating is on the basis of 120 hours operation, or 2,750 tons per week.

Refuse Characteristics

It would be convenient if incinerator furnaces could be rated in terms of heat units as is the practice with furnaces burning commercial fuels. This is not possible, however, except in a general way, because of the great variations in refuse characteristics, both from city to city and, in the same city, from day to day. These variations are caused by many factors, those which affect fuel value being briefly described as follows:

a) Rainfall affects the moisture content of refuse, which will therefore vary somewhat with the climate.

b) The extent to which coal is used for domestic heating, and the climate, both control the proportion of ashes in the total refuse.

c) Certain types of industrial refuse can obviously have a decided influence on fuel value.

d) Regulations as to separation by the householder of the several refuse components and the enforcement of such regulations will control the amount of non-combustible matter which reaches the incinerating plant.

e) Similarly regulations (and their enforcement) concerning the draining and wrapping of garbage and the quality of refuse containers and their covers affect the moisture content of the refuse.

f) The extent to which parts of the combustible refuse are diverted by salvage, garbage grinders, hog feeding, household burning and the like, is important.

g) Practice as to the extent of refuse collection service varies widely. In some cities, for example, service is given only to residences, thus excluding commercial refuse with its high proportion of combustible solids.

It should be noted that all except the first two of the above items are subject to control by the municipality.

It has been sufficiently demonstrated that the fuel value of municipal refuse, free of water and inert material or ash, is between 8,000 and 10,000 Btu per lb., with 9,000 a reasonable figure for design purposes. Thus the fuel value of refuse as received, depends on the proportions of water and of non-combustible solids. A rough indication of the fuel values of different classes of municipal refuse is given in Table 1. These values can vary over much greater ranges than shown, as they are affected by the factors listed above.

The effect of the differences in fuel values may be illustrated by experience at a plant whose furnaces were originally designed to burn combustible rubbish only at the rate of 16-2/3 tons per hour. Before the plant went into operation the municipality concluded not to separate the non-combustible portion of the rubbish and the plant has operated successfully burning mixed rubbish at rates up to 25 tons per hour. It would obviously be unfair to compare this plant with others on the basis of the original design without taking account of the class of refuse to be burned.

TABLE 1

Approximate Refuse Characteristics

Class of Refuse	Per cent water	Per cent in inert dry solids	Btu per lb.
Wrapped garbage	60-65	20	2,500 - 2,900
Combustible rubbish	20-30	15-20	5,000 - 6,000
Garbage and combustible rubbish	30-40	15-20	4,000 - 5,000
Combustible and non-combustible rubbish	20-30	50-60	2,500 - 3,500
All classes collected together	25-35	40-50	3,000 - 4,000

Furnace Ratings

The practice of rating incinerator furnaces on the basis of short period (usually 6 hours) tests derives from the competition among proprietary interests in several allegedly patented furnace designs up to the time of the second World War. During this period it was customary for a municipality to purchase incinerating furnaces under general specifications leaving the design to the furnace builder and relying upon guarantees to obtain the desired capacity. Compliance with the guarantees was determined by the short period tests, which could be manipulated to some extent and which usually resulted in furnace ratings that could not be maintained continuously over any considerable period.

In recent years the trend has been toward detailed design of the furnaces by the municipality's engineers, which has removed the necessity for guarantees and tests. It therefore behooves the engineer to take into account, in establishing criteria of design, the actual conditions under which the plant will operate.

For a plant which must incinerate all of the refuse of one or more classes, produced by a municipality, the logical basis for establishing required capacity is the average rate of production during the maximum week. This rate may be as high as 135 per cent of the annual average rate. Furthermore it may be necessary to operate at rates above 90 per cent of the maximum for as many as eight to ten consecutive weeks. This is illustrated by Fig. I, which shows the weekly variations in refuse quantities delivered to the incinerating plant at Winnipeg, Manitoboa, during 1957. This plant burns garbage and combustible rubbish, with good separation of non-combustibles by the householders.

The principal measures of furnace capacity are grate area and combustion volume. Grate area is usually taken as the total horizontal area of the furnace, and combustion volume includes both furnace chamber and combustion chamber. Because of constantly varying refuse characteristics and rates of

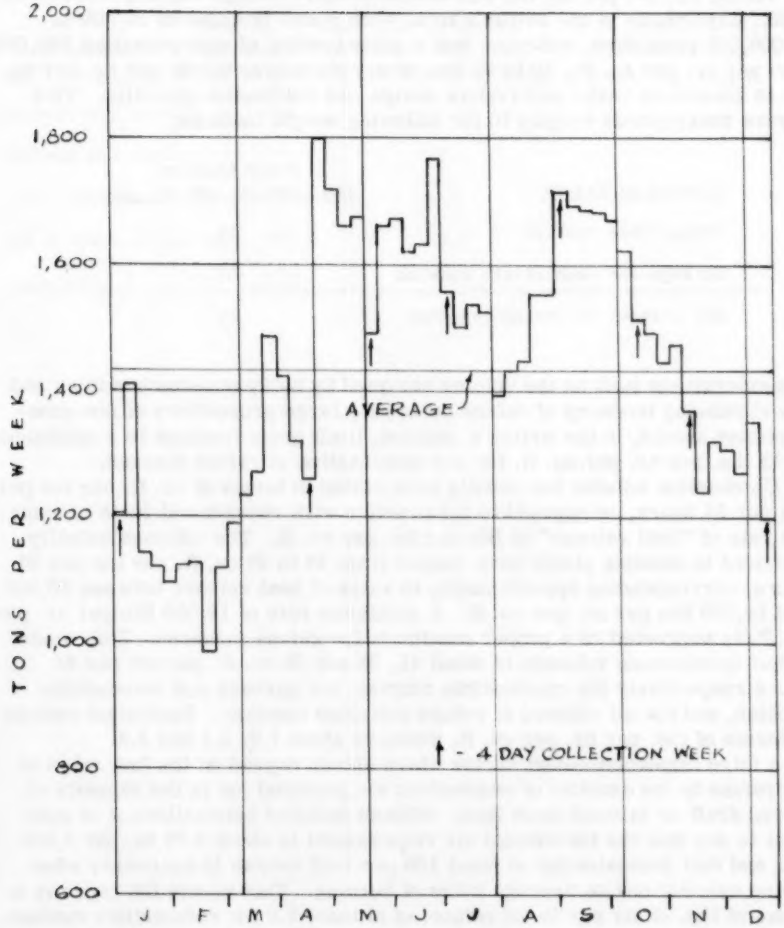

Fig. 1. Refuse Quantities Winnipeg - 1957.

burning (especially in batch-charged furnaces) refuse furnaces must be operated at average rates considerably below those considered normal for furnaces using commercial fuels. The high volatile content of the combustible solids also affects the combustion volume required.

Grate areas have commonly been rated in terms of the loading in pounds of refuse per hr. per sq. ft., with actual design loadings varying from 50 to 100. Experience in the writer's firm, with plants in cities of 50,000 to 1,000,000 population, indicates that a grate loading of approximating 300,000 Btu per hr. per sq. ft., 30 to 35 lbs. of dry inert-free solids per hr. per sq. ft. is consistent with conservative design and continuous operation. This figure corresponds roughly to the following weight loadings:

Classes of Refuse	Grate Loading Lbs. per hr. per sq. ft.
Combustible rubbish	50
Garbage and combustible rubbish	65
All classes collected together	75

Considerations such as the volume occupied by bulky non-combustibles and the clinkering tendency of refuse containing large proportions of non-combustibles should, in the writer's opinion, limit grate loadings to a maximum of 75 lbs. per hr. per sq. ft. for any combination of refuse classes.

Combustion volume has usually been stated in terms of cu. ft. per ton per ton per 24 hours, as opposed to the practice with commercial fuels of using the rate of "heat release" in Btu per hr. per cu. ft. The volumes actually provided in existing plants have ranged from 25 to 45 cu. ft. per ton per 24 hours, corresponding approximately to rates of heat release between 10,000 and 14,000 Btu per hr. per cu. ft. A maximum rate of 12,000 Btu per hr. per cu. ft. is suggested as a proper maximum for refuse furnaces. This would result in minimum volumes of about 45, 35 and 25 cu. ft. per ton per 24 hours respectively for combustible rubbish, for garbage and combustible rubbish, and for all classes of refuse collected together. Equivalent ratings in terms of lbs. per hr. per cu. ft. would be about 1.9, 2.4 and 3.3.

A third important design factor which should depend on the fuel value of the refuse is the quantity of combustion air provided for in the capacity of forced draft or induced draft fans. Without detailed calculations it is sufficient to say that the theoretical air requirement is about 0.75 lb. per 1,000 Btu, and that provision for at least 100 per cent excess is necessary when basing calculations on average rates of burning. This means fan capacity in terms of lbs. of air per lb. of refuse, of at least 9.0 for combustible rubbish, 7.5 for garbage and combustible rubbish and 6.0 for all classes of refuse together.

SUMMARY

1. Much confusion can be avoided if incinerating plant capacities are stated in tons per week, and if the unit of tons (or pounds) per hour is used for measuring furnace capacity. This practice would take account of the amount of refuse storage provided.

2. The class or classes of refuse for which an incinerating plant is designed should always be included in a statement of its capacity.

3. Design criteria for furnaces and for forced draft air fans are proposed in accordance with the following tabulation:

Class of Refuse	Furnace Loadings		Fan Capacity
	Lbs. of refuse per sq. ft. of grate area	Lbs. of refuse per cu. ft. of combustion volume	Lbs of air per lb. of refuse
Combustible rubbish	50	1.9	9.0
Garbage and combustible rubbish	65	2.4	7.5
All classes collected together	75	3.3	6.0

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DIVISION ACTIVITIES

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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SEWAGE DISPOSAL IN SANTA MONICA BAY, CALIFORNIA[a]

Discussions by Charles H. Lawrance, David R. Miller and D. I. H. Barr

CHARLES H. LAWRANCE,[1] and DAVID R. MILLER,[2] A. M. ASCE.—The author is to be congratulated upon his contribution to the knowledge of ocean disposal of sewage plant effluent. His paper represents a good condensation of the most important phases of the extensive offshore oceanographic and biological investigations which the City of Los Angeles accomplished recently supplementing their longer-established program of monitoring the Santa Monica Bay beaches for coliform densities. He is particularly to be commended in demonstrating the cardinal importance of T-90, the rate of coliform decline in sea water, and to show what wide variations may be experienced from one discharge condition to another with corresponding wide variations in overall coliform decline. Because of the dearth of field data regarding the operation of existing ocean outfalls, many of the factors discussed by the author were not fully appreciated until only very recently. The City of Los Angeles is to be commended for its whole-hearted efforts to solve its difficult sewage problem in the best interests of all concerned. The City's oceanographic program is an example of this endeavor.

The sewerage problems of the City of Los Angeles are perhaps unique. Few cities have grown at such a rate in the post war period and none has grown to such a population as has Los Angeles, whose sewerage system now serves an estimated 2,500,000 persons. Further, the Santa Monica Bay Area, which is bordered practically full length by extensive and extremely valuable bathing beaches has been and continues to be the logical disposal site for large volumes of treatment plant effluent, both from the economic standpoint and from other considerations. The burgeoning population of the Los Angeles Metropolitan Area makes increasing use of these beaches and the offshore water, and with the passage of time, increasingly higher standards for these receiving waters are being demanded by the public and required by State authorities. The completion of the existing plant in 1951 removed the unsatisfactory disposal conditions that had prevailed for many years and restored the bathing beaches to acceptable conditions. With the rapid development of the area, however, overflow conditions became experienced in the Los Angeles sewage collection system and the treatment plant approached its capacity sooner than had been expected. By the time a Citizens Committee had received and reviewed engineering reports on the possible courses of action for

a. Proc. Paper 1534, February, 1958, by C. G. Gunnerson.

1. Sr. San. Engr., Koebig and Koebig, Cons. Engrs., Los Angeles, Calif., formerly, San. Engr., Hyperion Engrs., Los Angeles, Calif.
2. Project Mgr., Daniel, Mann, Johnson, & Mendenhall, Cons. Engrs., Los Angeles, Calif. Formerly, Project Engr., Hyperion Engrs., Los Angeles, Calif.

the City and had made its recommendation, the sewerage conditions had reached an emergency state. The City foresaw that its own staff would be unable to complete soon enough the bulk of the major design required for relief of the situation, involving collection, treatment, and disposal facilities. The City therefore engaged a joint venture of private Los Angeles engineering firms* to perform this design work with the stipulation that the resulting design would meet all State and local requirements. The works designed by the engineers in connection with this program are as follows:

North Central Outfall	- 8 miles - 112" Diameter Trunk Sewer, mostly in tunnel.
Treatment Plant Modifications and Enlargements	- Expand the Hyperion Treatment Plant to 420 m.g.d. average flow capacity and make certain modifications.
Ocean Outfall for Sludge Discharge	- 22" Steel outfall running 7 miles to sea.
Ocean Outfall for Effluent Disposal	- 144" Concrete outfall running 5 miles to sea, terminating in a major diffuser.
Effluent Pumping Plant	- Pumping Station - 720 m.g.d. capacity.

The basic premise used in this program was that the ocean would be used as a means of secondary treatment as well as disposal and that the treatment would consist of coarse bar screens, aerated grit removal, and primary sedimentation before disposal to the effluent outfall.

This effluent outfall was of unprecedented magnitude, and the timetable which was involved permitted and required thorough investigations before design was finalized. Therefore the engineers undertook an exhaustive research and investigation program dealing with the problems incident to the disposal of primary effluent into the ocean. The oceanographic studies described by the author were one of the phases of this program. In order to insure that all of the aspects of the program were thoroughly explored, the engineers retained the Allan Hancock Foundation of the University of California to undertake a one-year study of the oceanographic and biologic conditions existing in Santa Monica Bay. In addition, a staff program was initiated to study the performance of existing outfalls, and other research programs were authorized. This material was correlated with the studies described by the author and then the two outfalls were designed by the engineers.

It is anticipated that one or more papers will be prepared in the near future describing these other studies and the final designs. The following paragraphs will offer comments on certain portions of the author's paper and are made in the light of the other studies previously described.

The point that the author makes that the physical dilution of an effluent discharged into marine waters is ultimately limited by the amount of new water which is brought into the area should be particularly emphasized in the case of large quantities of effluent discharged in regions where the natural currents

*Hyperion Engineers: Consisting of Holmes & Narver; Daniel, Mann, Johnson, & Mendenhall; and Koebig and Koebig.

tend to be sluggish. However, the difference in density of effluent compared to sea water will, in itself, generate minor currents. The minimum dilution that could be expected even under quiescent conditions would be one in 60 for the proposed effluent diffuser. However, the point is that in the design of an ocean outfall there is no substitute for a comprehensive study of long term current and temperature conditions in the vicinity of the outfall in order to be able to predict with reasonable assurance the action of a large scale diffuser.

In regard to the matter of the coefficient of eddy diffusion, it should be remembered that the bacteriological factors far over-shadow the physical, particularly as time increases, following initial discharge.

The writers like one aspect of using the formula for the persistence of coliform bacteria in sea water:

$$\frac{N_t}{N_o} = 10 - \frac{t}{T\text{-}90}$$

The use of N_t = number of survivors after t hours makes it difficult to apply the formula without having current studies, which cannot be emphasized too much in studies of this kind. Also, suffice it to say that coliform bacteria are not the most reliable indicator of sewage pollution, and certainly further research in this field is needed. Coliform bacteria tests should be used with caution. It is essential that statistical methods be used in the interpretation of test results.

The thought that the waves or surf breaking up clumps of coliforms might alter the apparent MPN could be misleading. The coliform test procedure calls for a vigorous shaking of the sample bottle and if followed would be expected to simulate the effect of the more common waves or surf. Admittedly, large breakers would produce a greater turbulence than bottle-shaking.

The author states in his summary that the reduction of coliform density for primary effluent (T-90) is 3.4 hours. The engineers did not use this value in the design of the effluent outfall; rather they used a figure of 8 hours as a matter of conservative practice.

The conclusion of the author that a higher degree of treatment results in an effluent less amenable to ocean disposal sounds paradoxical, but the results of studies that were undertaken for the engineers would support this conclusion.

Separation of the major factors which combine to produce a reduction of chemical and bacterial pollutants following injection of sewage effluent into sea water is of particular concern for a specific discharge condition under study and is valuable from the standpoint of basic knowledge. This analysis has been handled very adeptly by the author in his studies in Santa Monica Bay, where he has shown that for high-rate activated sludge effluent the elements of sedimentation, physical dilution, and mortality were approximately equal for reduction of coliform densities. The engineers found it possible to approximate the separate effects of certain factors responsible for coliform decline in some of the data analyzed in their studies using somewhat different methods.

For example, in the 1936 data for the Deer Island Outfall in Boston Harbor* where coarse-screened sewage was discharged through a small diffuser with

*Massachusetts Department of Public Health, House No. 1600, Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor, December, 1936.

average port submergence of 43 feet at mean low water, the physical dilution was computed from recorded chemical data and also by a formula developed by Norman H. Brooks, J. M., ASCE.* The respective results were in fair agreement. The eddy diffusivity employed in the formula was

$$K = 0.01\ L^{3/4} \quad cm^2/sec.$$

as compared to the value of $0.005\ L^{3/4}$ which the author determined from the mildly turbulent Santa Monica Bay. The average current was taken as 0.97 foot per second and the average initial dilution as 23 parts of diluted effluent per volume of undiluted effluent. The original measurements were made from surface samples taken at intervals from the approximate center of the sewage field near current floats, as the field moved away from the diffuser from distance and time up to about 16,500 feet and 4.7 respectively. Although the data were somewhat erratic, physical dilution of the sewage effluent beyond the outlets did generally increase and was, for example, about 140 after 13,000 feet and 3.7 hours of travel. From data such as this it was determined that the decline rate of sewage bacteria attributable to all causes except turbulent diffusion was approximately 4.4 hours for 90%decline. The overall decline rate of sewage bacteria including the dilution effect was determined graphically from the bacterial data as about 2 hours for 90%decline. Thus the non-dilution effects were responsible for approximately 55% of the overall bacterial decline, since

$$\frac{1}{2} = \frac{1}{4.4} + \frac{1}{3.66}$$

$$\text{and } \frac{1}{3.66} \times 1/(1/2) = 55\%$$

or dilution effects and non-dilution effects following initial discharge were approximately equivalent in causing bacterial decline.

For Deer Island Outfall and the Moon Island and Nut Island Outfalls, a substantial portion of the dissipation of the field was vertically downward by the sedimentation of the heavier particles of suspended solids, accompanied by many bacteria associated with particulate matter. This is attested both by the results of subsurface water sampling carried on in some of the studies and by the high bacterial concentrations found in the bottom muds of the harbor.

A fairly striking example of the effects of sedimentation appeared in the recent offshore studies of the Allan Hancock Foundation** over the Orange County Sanitation Districts Outfall, which is a 78-inch diameter pipe extending

*The reference in which this formula first appeared is "Appendix A, Methods of Analysis of the Performance of Ocean Outfall Diffusers With Application of the Proposed Hyperion Outfall, April 3, 1956, an appendix by Norman H. Brooks, Consulting Engineer, to accompany Hyperion Engineers Report Relating to Length of Outfall and Types of Diffusers for the Ocean Outfall for Effluent Disposal, April 6, 1956." This formula is also discussed in "Ocean Outfall Design, Oct. 15, 1957, Los Angeles, California, Hyperion Engineers, a Joint Venture."

**Allan Hancock Foundation, University of Southern California, The Distribution of Coliform Bacteria in the Waters Surrounding the Orange County Sewer Outfall, March 2, 1956.

about 7,000 feet into the Pacific Ocean and terminating at bottom depth of about 55 feet in a small diffuser yielding an initial dilution of about 26. From observations of coliform densities taken in depth in the vicinity of the outfall on four separate cruises in the autumn of 1955, it could be noted that as one moved away from the outfall outlets, the surface coliform densities declined but that subsurfacely a zone of higher coliform density developed and extended outward like a "tongue" before diminishing. Chlorinities and temperatures were also determined and showed that the sewage-sea water mixture remained near the surface above this subsurface tongue which was entirely within water characteristic of the normal sea water in the area.

The generalized pattern of these observations is depicted in the bacterial profile of Fig. A. The general configuration of the coliform contours showed a coliform density in profile which subsided roughly at a slope of 10 feet vertical depth per 600 or 700 feet horizontal distance or, at an average current of 1000 feet per hour, at a rate of about 13 feet per hour. The overall decline rate of coliform organisms at the surface may be approximated by noting the distance from the outlets required per magnitude (90%) reduction in coliform density and applying the average current of 1000 feet per hour. When this is done it can be seen that values of T-90, after an initial value of about 2.0 hours, tend to average about 1.3 or 1.4 hours, which is close to the 1.5 hours overall average reported by the Hancock Foundation and depicted in the author's Fig. 6.

The subsurface coliform decline rate, slanting downward from the surface above the outlets and following the subsurface tongue, proceeds much slower than the surface rate and initially is in the order of 3.3 hours per magnitude, speeding up appreciably after about 3.3 hours have elapsed. Because the physical and chemical characteristics of these subsurface waters were non-sewage in nature, it could be inferred that a substantial portion of the coliform population present was associated with particulate matter which settled out of the surface layer of diluted sewage.

The profiles represented in Fig. A were run generally parallel to the coastline and hence sedimentation of particulate matter and bacteria was not ultimately impeded by surf conditions as in the generalized case described by the author for Santa Monica Bay. The data of Fig. A and those of individual cruises characteristically showed coliform sedimentation. It is of interest to note that the primary effluent discharged from this outfall originates at two treatment plants where the primary sedimentation overflow rate was in order of 800 and 2200 gallons per day per square foot, respectively or 1500 on the average corresponding to 8.3 feet per hour settling rate in fresh water. It might appear at first glance that the settling rate of the particulate matter in the primary effluent in the sea water after initial dilution was approximately 13 feet per hour, corresponding to the general slope of those coliform contours closest to the outfall as previously mentioned. However, the factors of dilution and mortality both tend to obscure the true settling rate of the bacteria measured by the concentration of the bacteria themselves. Also of importance with this regard are these factors: (1) The horizontal currents tend to vary both in magnitude and even in direction as one moves from the surface towards the bottom. Usually the current magnitude decreases with depth and subsurface currents at depth may sometimes be at an appreciable angle to those at the surface. For this reason the path of a settling particle would probably curve downwards and perhaps even laterally as opposed to the two-dimensional slanting line of a settling particle in an "ideal" settling tank.

(2) The settling particles are heterogeneous and there is probably a distribution of settling rates among the various particles. For these reasons it is impossible to isolate the separate elements of sedimentation, dilution, and mortality without resorting to certain generalizations and speculations. The following illustration employs such a premise.

The maximum depth of the sewage field was found by chlorinity measurements to be about ten feet. Often the field was much shallower. The physical dilution in this surface water was computed both by chlorinity measurements and Dr. Brooks' formula, and the two methods gave fair agreement. The physical dilutions were found to be approximately 1.5 times S_o (initial dilution) after 1 hour, 2.0 times S_o after 2 hours, 2.5 times S_o after 3 hours, 3.5 times S_o after 4 hours, and 6.0 times S_o after 6 hours. The chlorinity measurements were from surface samples taken within the sewage field "tagged" by sea-marker dye as it moved away from the outlets. These "dye patch" experiments were made on five separate days, covering a total of 34 samples, and covered intervals of 1 hour, 2 hours, 3 hours, 4 hours, and 6 hours away from the "boil" above the outlets. Coliform determinations were also made and were the basis of the value of the T-90 of 1.5 hours referred to by the author. Table 1 summarizes some of these dye patch experiment results as analyzed by the engineers.

The values of N_o/N_t in Table 1 expressing overall effects of coliform reduction measured by coliforms represent geometric means of the individual determinations. Among the values listed, those for 2 hours and 4 hours time have the most number of samples and are presumably the most reliable while those for 3 hours and 6 hours have the fewest samples and are probably the least reliable. A statistically significant difference among the T-90 values of a given category for differing times would be hard to establish, yet except for the 6th hour the patterns for all categories are quite consistent and show a more rapid reduction of coliform organisms in the first hour by dilution effects, non-dilution effects, and overall causes, than for subsequent hours, particularly the third and fourth hours. It must be that at least a portion of this effect is caused by the phenomenon of sedimentation of the bacteria, the bulk of which would be expected to occur fairly early in the life of the sewage field. Admitting this, it will follow that the non-physical dilution effects for the later hours of life of the field, typically three hours and four hours (assuming the six-hour condition is for some reason non-representative) would represent mortality alone, or $T\text{-}90_m$ would be perhaps 2.3 hours.

Table 1 Dye Patch Experimental Data, Orange County, California, 1955

Time	Overall Effects By Coliforms			Physical Dilution		Non Physical Dilution Effects	
Hours	No. Samples	N_o/N_t	T-90	S_t/S_o	T-90d	N_o/N_t	T-90
1	6	5.16	1.40	1.5	5.69	3.44	1.86
2	10	22.0	1.49	2.0	6.67	11.0	1.92
3	4	54.7	1.73	2.5	7.54	21.9	2.24
4	10	187	1.76	3.5	7.36	53.5	2.32
6	4	13,800	1.45	6.0	7.70	2300	1.78

Now during the first hour, bacteria which have settled out of the stratified sewage field at the median rate will generally have been reduced in volumetric concentration only by the action of dilution and mortality. First hour dilution accounts for a dilution ratio of 1.5, and this would apparently hold no matter how fast the bacteria settled provided they dropped out of the surface field in the first hour, for which there is indication. The assumed mortality rate of 2.3 hours per magnitude corresponds to an "apparent" dilution ratio of 2.7, resulting in an overall dilution ratio

$$\frac{N_o}{N_t} = 1.5 \times 2.7 = 4.05$$

This value would represent the concentration of typically-settling coliforms after one hour's time as compared to their initial value in the "boil". The surface concentration after one hour, on the other hand, has received an overall "apparent" dilution of 5.16 times over the "boil" concentration. Under these approximations and suppositions therefore, first hour sedimentation would have accounted for

$$\frac{(5.16 - 4.05)}{4.05} = 27.5\%$$

say between 25 and 30 per cent of the overall first hour coliform decline.

After the first hour or actual time required for the average settling bacteria to drop out of the sewage field the effect of dilution would be curtailed because of decreasing turbulence and the overall decline rate would be proportionately slowed down, hence the tendency for the subsurface "tongue" of Fig. A to persist.

One point not stressed by the author yet tacitly contained within his presentation is the great effort and expense required to obtain significant sampling data, especially offshore data. The surf sampling program alone has involved thousands of samples each year with concurrent field, laboratory, and statistical work. Offshore sampling requires an oceanographic vessel with navigational and laboratory equipment. Operation of such a vessel is in itself expensive and when the ever-changing waters of the ocean are to be studied to any significant degree, the requirements for time and money are magnified.

A further difficulty in the quest for the all-important data regarding bacterial decline in sea water for areas such as Santa Monica Bay is the inability to perform measurements except under very limited circumstances. This is because the ever-rising standards for water quality for Santa Monica Bay generally preclude the cessation of chlorination of plant effluent for bacterial reduction. A few years ago, the standards in force did not require that the beaches and offshore waters receive full protection during the winter months, and therefore chlorination was not practiced during these months except when warm weather was forecast and many bathers were expected. This condition no longer holds and chlorination is practiced continually on a year round basis with the obvious consequence of preventing plant-scale experiments in decline of unchlorinated effluent in the ocean. During the design of the proposed 5-mile Hyperion Outfall for effluent disposal there were only four occasions for which the chlorination at Hyperion was intentionally interrupted, and these intentional interruptions ranges from 10 to 24 hours in duration. One of these occasions was the radioactive tracer experiment of May 22-23, 1956 in which SC-46 was discharged with the effluent and subsequently measured in the

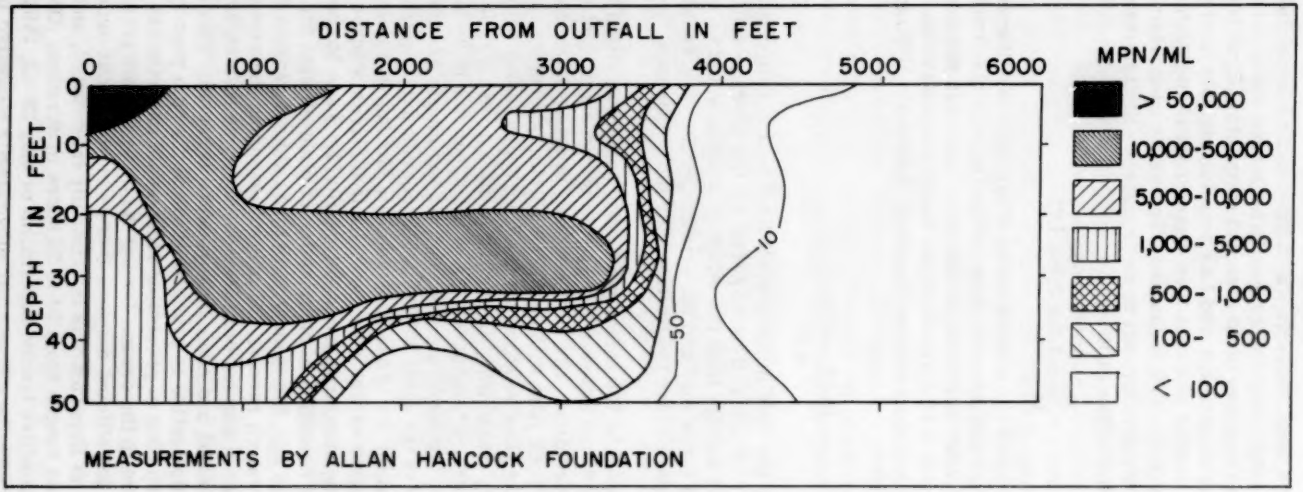

FIGURE A
VERTICAL DISTRIBUTION OF COLIFORMS INDEPENDENT OF DIRECTION IN THE WATERS SURROUNDING THE ORANGE COUNTY SANITATION DISTRICTS SEWER OUTFALL DURING THE AUTUMN OF 1955

sewage field and receiving waters. Special advance permission had to be obtained from State authorities before this experiment could be performed.

The extensive surf sampling experience for Santa Monica Bay augmented the more limited offshore studies in the Bay and these were supplemented by the offshore studies off Whites Point and Orange County alluded to by the author.

D. I. H. BARR,[1] A. M. ASCE.—In considering the hydraulics of dilution of sewage effluent released off-shore, the author tentatively divides dilution subsequent to that caused by diffusers into two separate phenomena; turbulent mixing due to density variations and lateral mixing due to eddy diffusion. Both the terminology and the tentative assumption seem to require further consideration.

Since the term eddy diffusion is intended to describe the process analogous to diffusion due to turbulent mixing, in which separation between the vertical and lateral coefficient of diffusivity has been noted as being mainly due to density stratification, (ref. 6), it is not obvious why 'turbulent mixing' and 'lateral mixing' are separated in the paper.

Regarding the dilution process it would seem more correct to say that dilution is caused by the two phenomena, density currents and turbulent mixing. Considering the details given of dilution at the existing Hyperion outfall, local horizontal variations in density would give rise to irregular mass-transferring density currents superimposed on the general bay current and causing turbulence over and above the general turbulence. The level of turbulence would appear to affect the dilution and thus the depth of the density layer, whether on the surface or at intermediate depth, rather than its extent. Subsequent to the disappearance of local density differences, density difference would appear to have a continuing effect on the spread of the contaminated zone. The average ratio of spread on one side of the test dye streams to travel length is about 1 to 30 and for the isolated maximum is about 1 to 11. (Fig. 3). But in Fig. 2, where one side of the sewage effluent zone is bounded by the shore, the ratio of spread to travel length of the other side is about 1 to 6. If the density of the sewage effluent approximates to that of fresh water, as compared with sea water, the dilution of 1 part per 314 parts sea water, given as typical after 24 hours, would cause a density difference of about 0.00009 gm. per m.l. Although small, this could cause a spread of several thousand feet in 24 hours starting from a statified area of a few feet depth. Precise figures of densities, etc. and details of local conditions are not available to the writer, but the author's comments on this hypothesis would be of interest. The mechanics of lateral spread of non-homogeneous water with a slight density difference for both the cases where non-homogeneous stream is moving with the main body of water and for the case where it is moving through the main body of water, do not appear to be fully investigated.

Can the author give any details of the effect or lack of effect of surface waves on dilution from his observations.

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DIGITAL COMPUTERS FOR PIPELINE NETWORK ANALYSIS[a]

Discussion by M. B. McPherson and J. V. Radziul

M. B. McPHERSON,[1] A. M. ASCE and J. V. RADZIUL.[2]—The authors have given a succinct description of both their program and the manipulative functions of the digital computer. Their list of references is indeed generous. As they have suggested, a study of their program should facilitate the understanding of digital computers "in the solution of other engineering problems."

This paper and a paper by the writers(1) were presented at the same ASCE Convention session and appeared in the Proceedings simultaneously. The paper by the writers included a critique of the merits of different types of computers in water distribution design from the standpoint of use by both larger utilities and consultants. The need expressed in the following sentence, from the above reference, has been satisfied explicitly by the authors' computer program: "As soon as a standard set of instructions is published (particularly for the stored-program type of computer), together with an outline for setting up data for problem mains, loops and special operating instructions, the digital computer will become an extremely valuable design tool for all consultants." The "Flops" system is an added bonus feature.

The writers are aware of some programs in which a 'hunting' or a hysterisis effect occurred with successive iterations, resulting in erratic relaxation, or convergence, of the final results. It is assumed that the program given as well as the "more general program for extensive water distribution systems ------not included here-----" do not display these characteristics. It would be helpful if the authors would cite an example of the number of iterations required to achieve a specified head loss tolerance for a given number of mains and loops, as compared with the information given by Hoag and Weinberg.(2) The final answers in Fig. 2 suggest that a head loss of 0.1 ft. might have been the criteria for ceasing iterations. Although not mentioned, it is important to note that the initial, assumed flow rates need not necessarily be more than a guess as to magnitude and direction. However, the number of iterations required is related somewhat to the diversity between the initial and final flow values. The use of the term "K" should be clarified: in Table 4 it is for a 1000-foot length; in Tables 2, 3 and 5 it is for the total length of pipe (i.e., a, b, c, etc.). The manner in which the sign (flow direction) is accounted for and carried through in the sequential steps of Table 5 is not clear to the writers. To facilitate an evaluation of the computations in terms of physical features,

a. Proc. Paper 1608, April, 1958, by Quintin H. Graves and Don Branscome.
1. Research Engr., Philadelphia Water Department, Philadelphia, Pa.
2. Water Distribution Design Engr., Philadelphia Water Dept., Philadelphia, Pa.

it would be appreciated if the authors would add the lengths, diameters and "C"-values to Fig. 1 (or Fig. 2) in their closure to the discussions.

Demountable scales are used in conjunction with a unit-scale master voltmeter and ammeter for reading directly head loss and flow with the McIlroy Network Analyzer. A variety of scale ranges are provided and the scales themselves are available in any desired units of flow and head. The range of head loss or flow for a given scale is established by setting a four-position shunt. Hence all measurements are made directly in flow and head loss units. The authors mention repeatedly their erroneous conception: "These values (measurements) must be reconverted into the corresponding hydraulic terms."

The authors persist in pitting the McIlroy Analyzer against the digital computer when there is little equitable basis for comparison. As opposed to the multitude of various types of digital computers now in use (there are an astounding number of '650' machines, alone), there are installed at present only 15 McIlroy Analyzers. Of the fifteen, seven are exclusively employed on gas network analyses, one is used for air and only three for water. Only the remaining four machines are normally available to the public on a rental basis. The McIlroy Analyzer is a direct-analogue, direct-reading, specialized design tool without peer, not merely a calculator or a computer (when full advantage is taken of all its features). The cost of a medium-sized McIlroy Analyzer is comparable to that of the newer types of desk-model digital computers, which, as the authors have emphasized, are not restricted to the "one special purpose of solving fluid network problems." It is fairly obvious that the McIlroy Analyzer is particularly suited to those owners who have a full-time use for the machine and who can profit from the full utilization of its features in making rapid design decisions—namely the larger utilities. The writers have observed instances in which the calculations for a complex system were made on a digital computer where the 'solution' proved no more than the fact that the system could not be operated under the conditions prescribed. This was not obvious until practically all the readouts had been summated. Using a McIlroy Analyzer, the impossibility or impracticability of a design arrangement is usually evident before any readings are even taken and often before the network is brought up to full load, due to the complete analogy which the machine affords. In a typical design a multitude of alternates and modifications must be studied. Often, a single head loss reading will suffice for an intermediate design step.

The statement "The engineer interested in the solution of the problem must spend considerable time in arranging his data for setting up the equipment on the (McIlroy) analogue computer and in converting hydraulic units into electrical units" etc., is incorrect. The designer need have no connection with the simple programming, set-up or physical operation of the machine, just as he would not be required to personally program, punch cards or tape and operate a digital computer. To the contrary, the designer's participation should commence with the operation of the machine. If the designer is not present during the tests, the McIlroy serves only as a computer; in his presence it becomes an easily manipulated 'model' of the system under study. For example, should he wish to see the overall effect of redistributing the proportion of demand between two sources (pumps or storage) this may be achieved by adjusting two rheostats and the overall result is immediately at hand. With a digital computer, new corresponding data cards must be punched, all the data cards run through the computer and the new answers summated before comparable results can be evaluated. (The authors are correct insofar as the need for a

specialist to program and operate the machine is concerned, but only one specialist is necessary).

For a network comprising no more than about ten interior loops, head loss calculations should be performed with equal facility using either a McIlroy Analyzer or a digital computer. In experiences with a complex gas network with 27 interior loops and 72 mains, using both types of machines, no particular difficulty was encountered in the McIlroy Analyzer tests. However, the myriad details inherent in the solutions with the digital computer became oppressive; one of the solutions was the inoperable case previously cited.

While it is true that simple networks can be calculated by hand ("The claim is made that high school students under supervision can turn out the required arithmetic very economically"), the typical complex networks which are studied with digital and McIlroy machines would require so much time if calculated manually that the designer might well lose sight of some of the several original objectives by the time the calculation was finished. It should be noted that satisfactory convergence by hand calculations may be practically impossible, depending upon the complexity of the network.

The feasibility and ramifications of analyses with balanced pump and equalizing storage characteristics have been outlined,[1] and recently an example of an hour-by-hour pump-network-storage balance was presented.[3] Due to the many adjustments necessary in storage balance procedures, the McIlroy Analyzer would normally provide a quicker, less complicated series of answers. This is particularly true for complex networks.

REFERENCES

1. McPherson, M. B. and Radziul, J. V., "Water Distribution Design and the McIlroy Network Analyzer," Proc. ASCE, Paper No. 1588, April, 1958.

2. Hoag, L. N. and Weinberg, G., "Pipeline Network Analysis by Electronic Digital Computer," Jour. A.W.W.A., p. 523, May, 1957.

3. McPherson, M. B. and Radziul, J. V., "Some Distribution Design Considerations," a paper presented to the Penna. Section, A. W. W. A., at Erie, Pennsylvania, on June 26, 1958.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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c. Discussion of several papers, grouped by divisions.

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